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JULY, 1954. Vol. XLIX, No. 7

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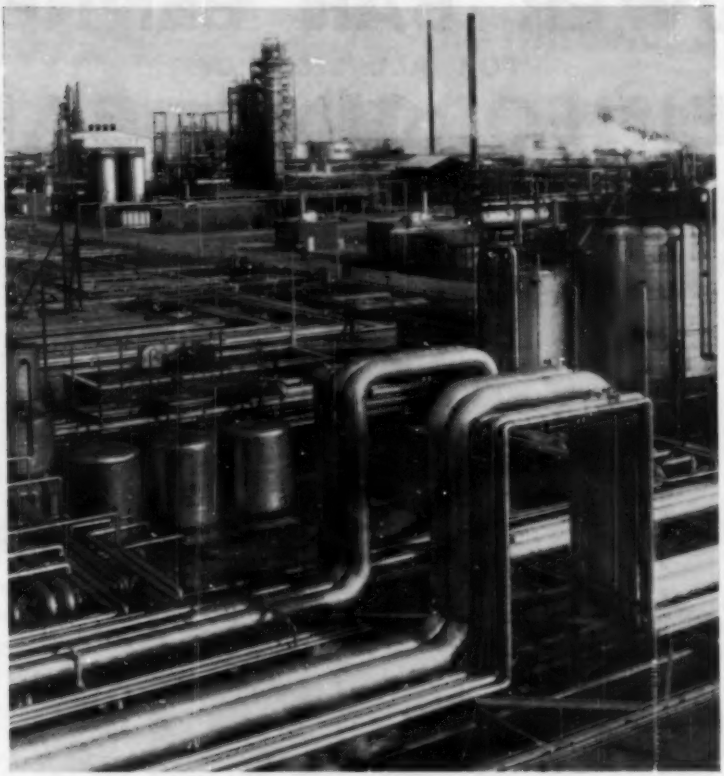
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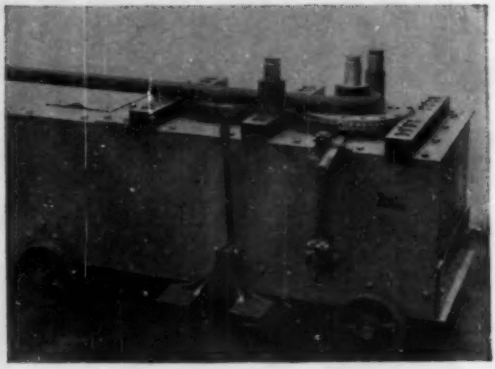
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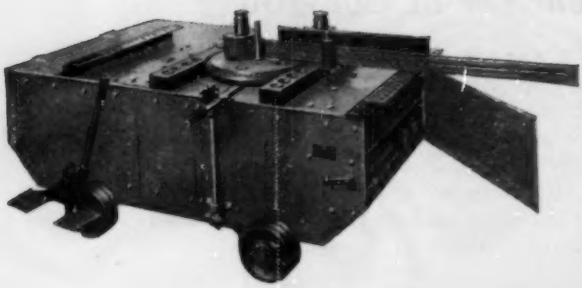
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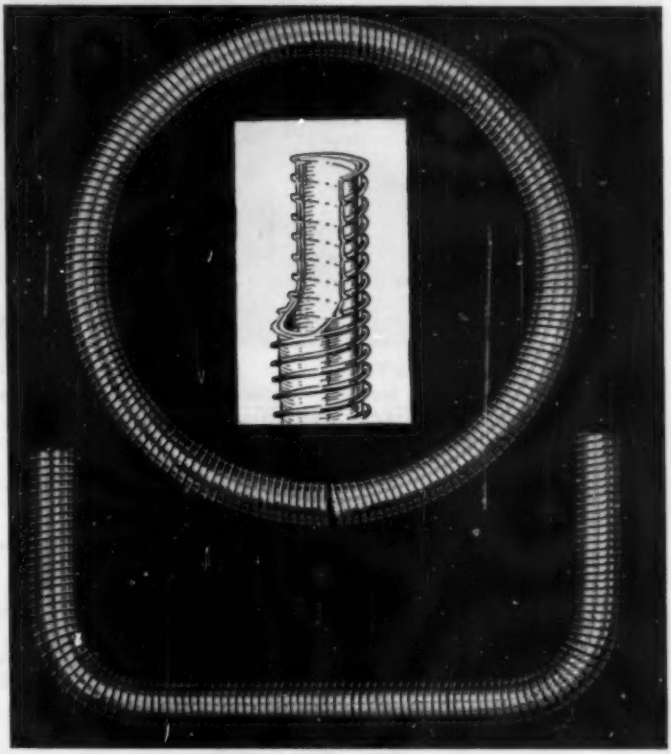
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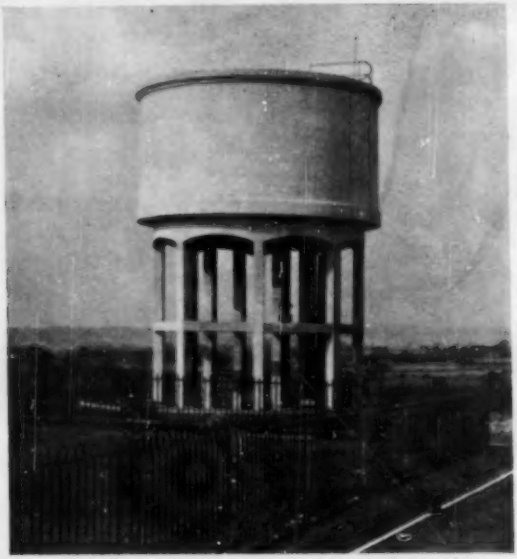
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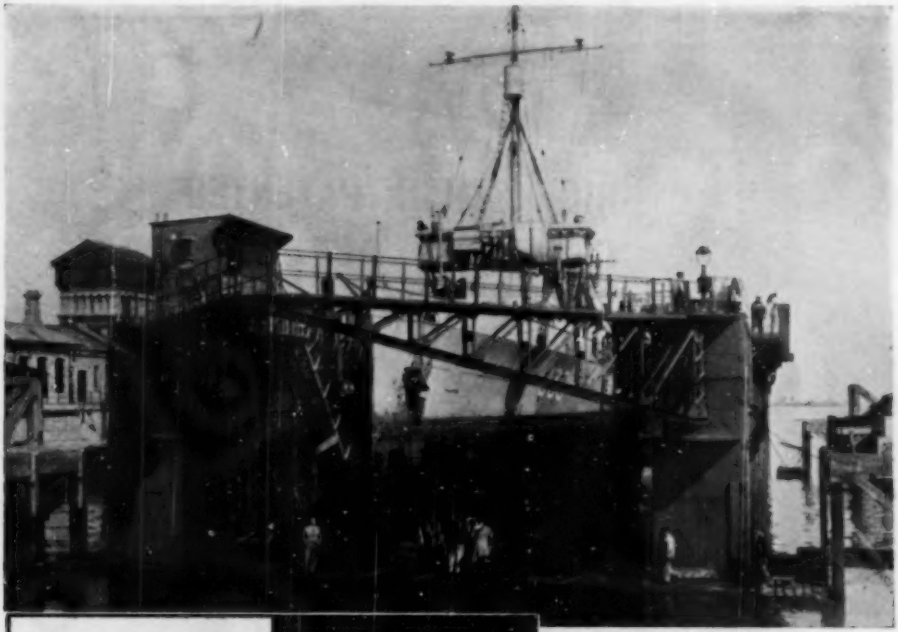
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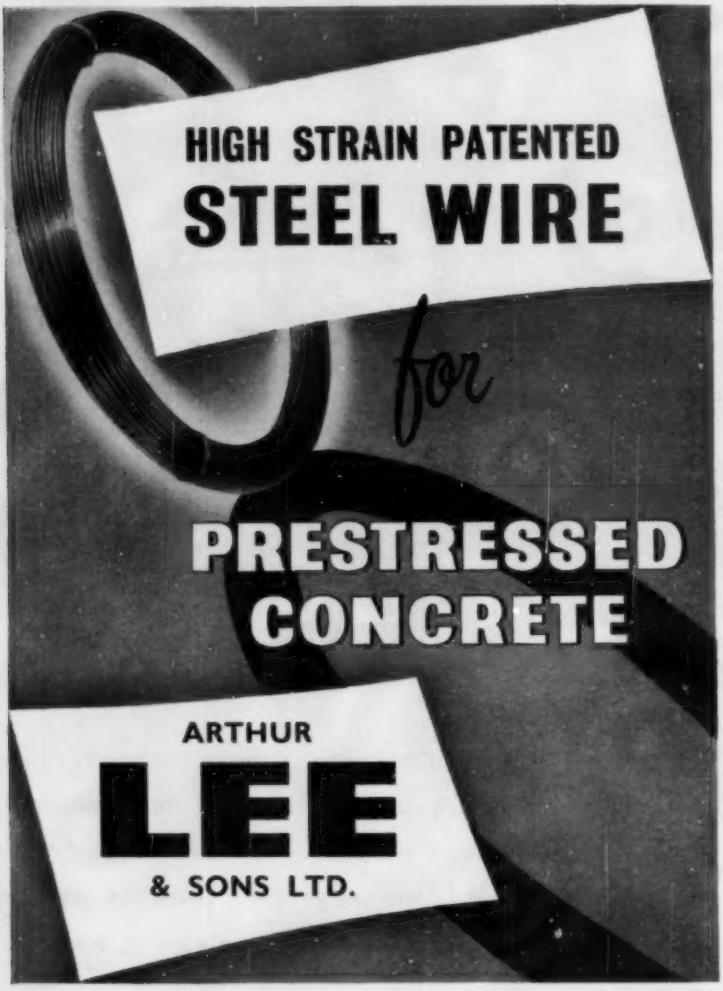
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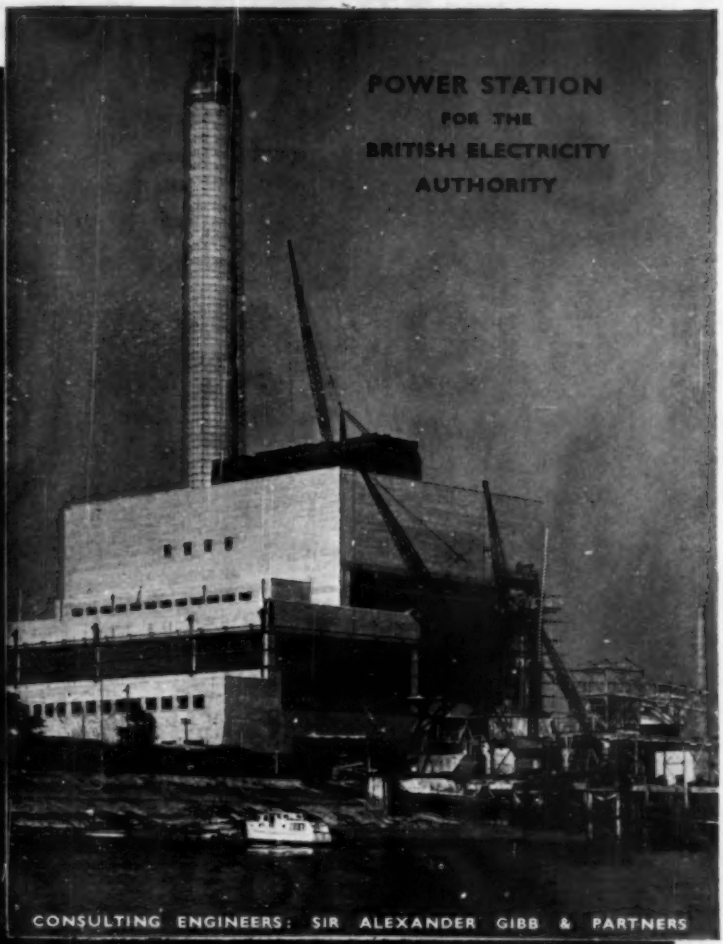
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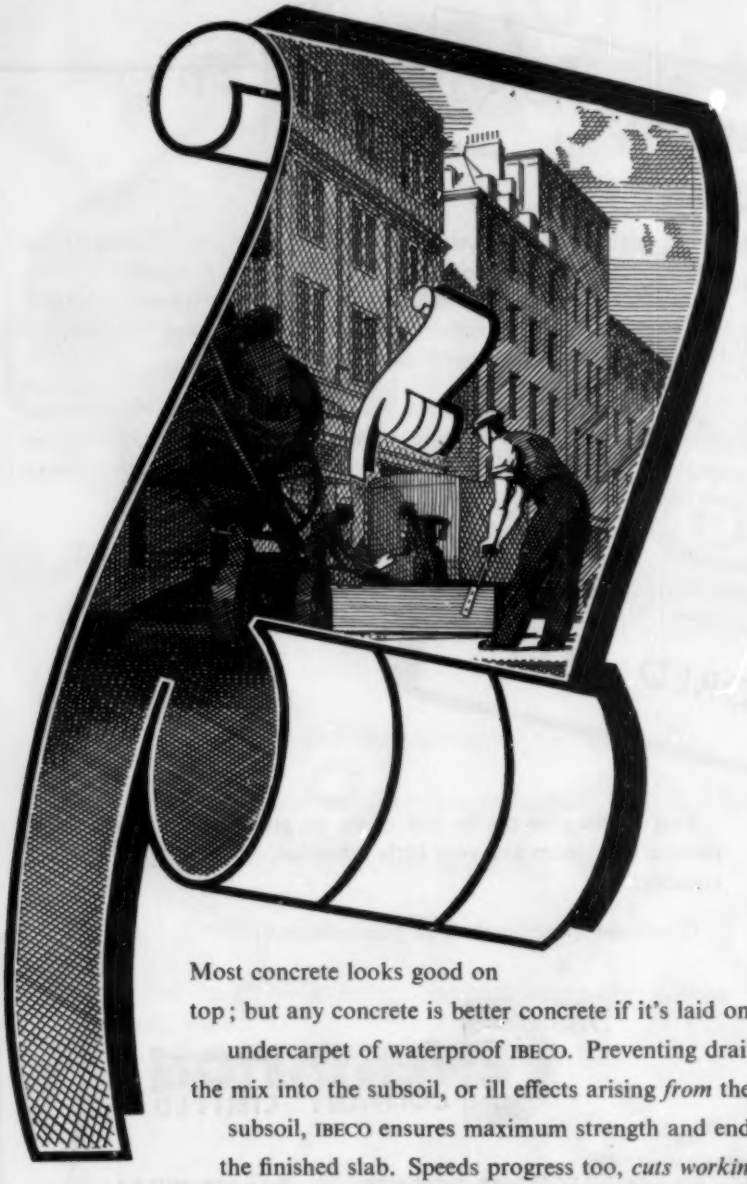
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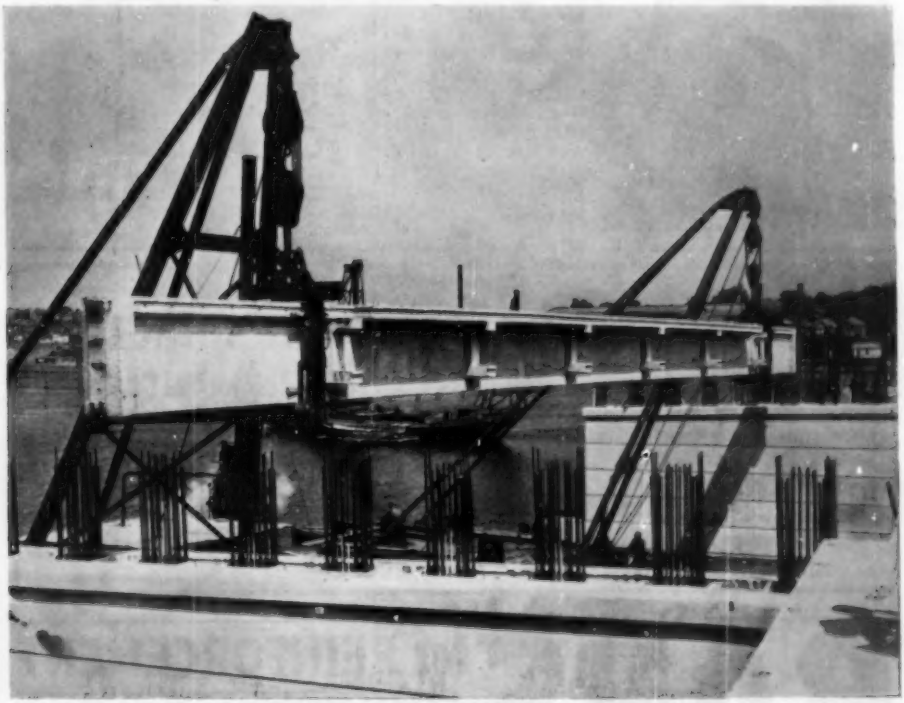
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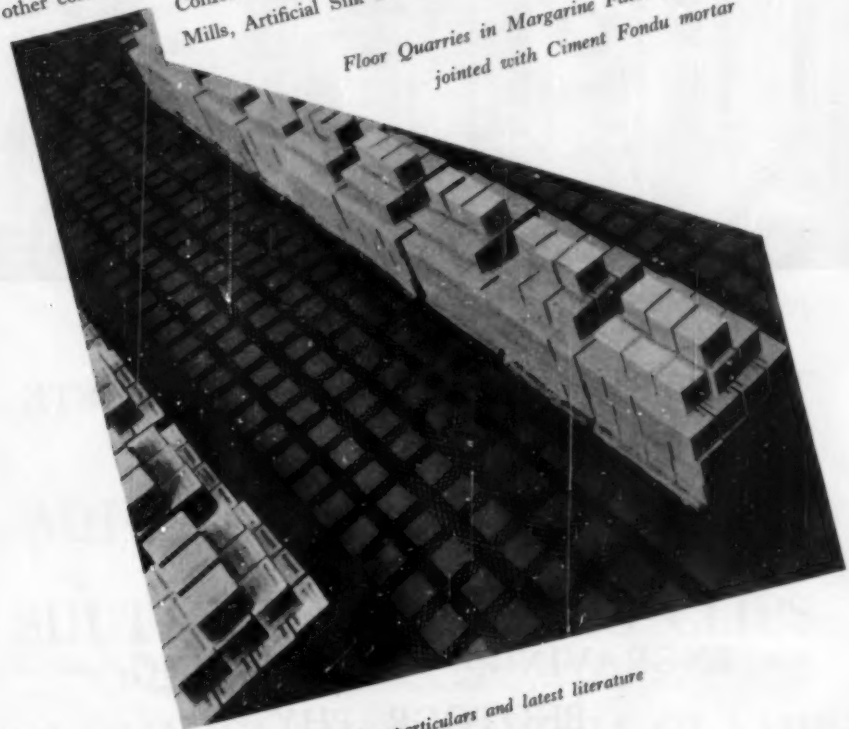
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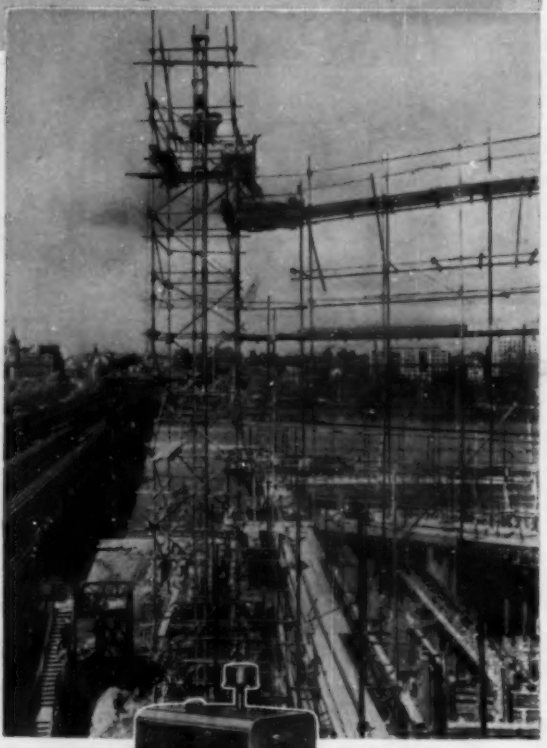
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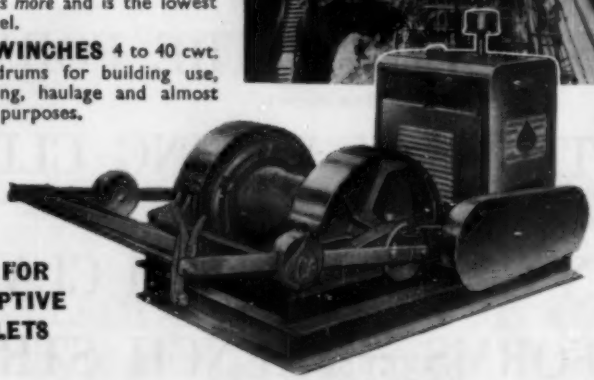
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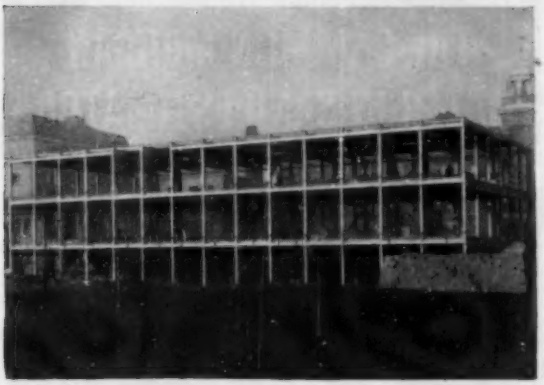
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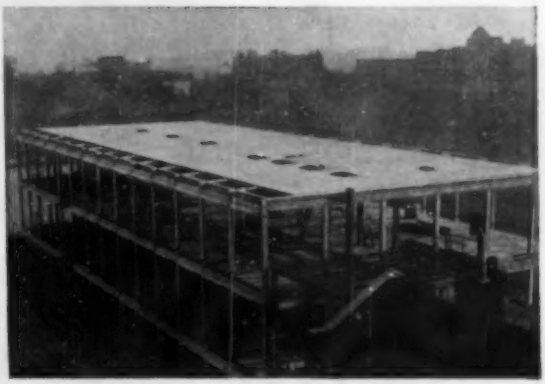
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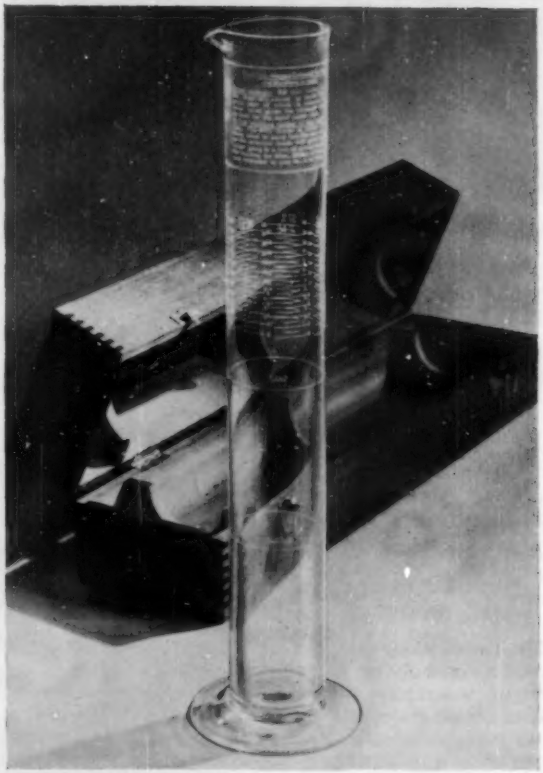
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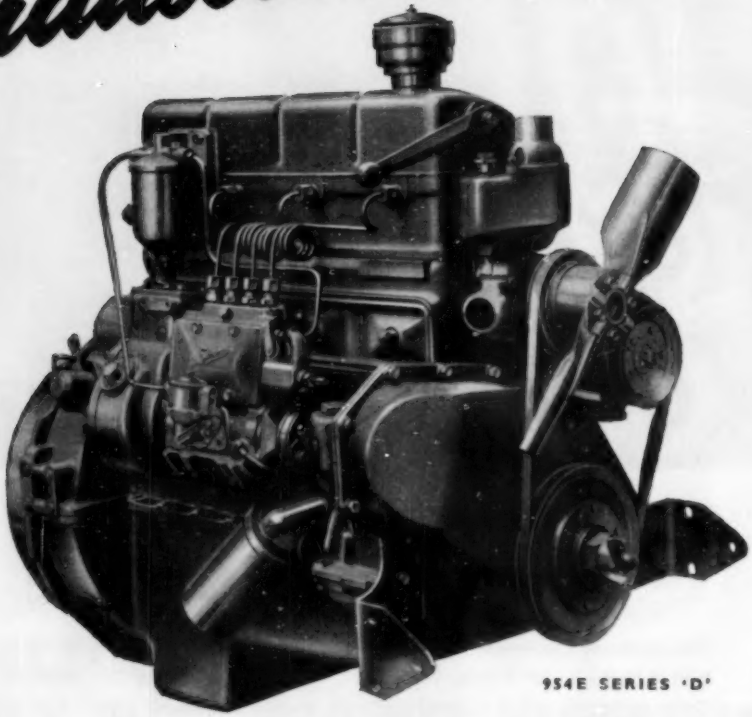
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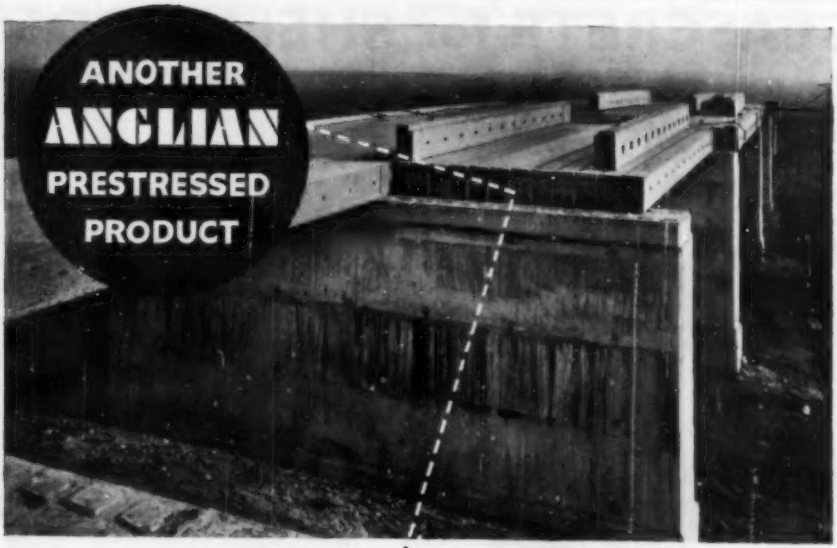
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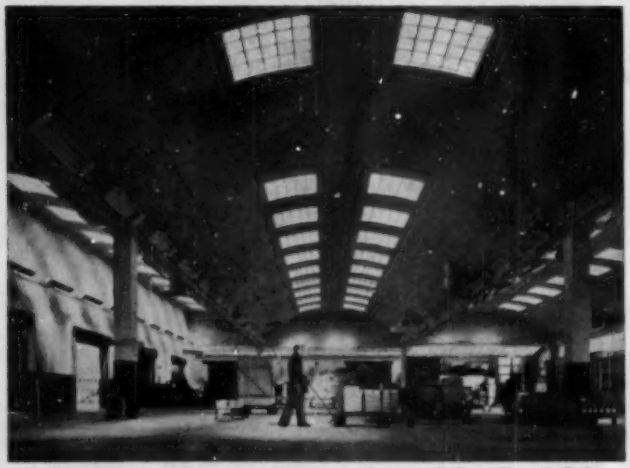
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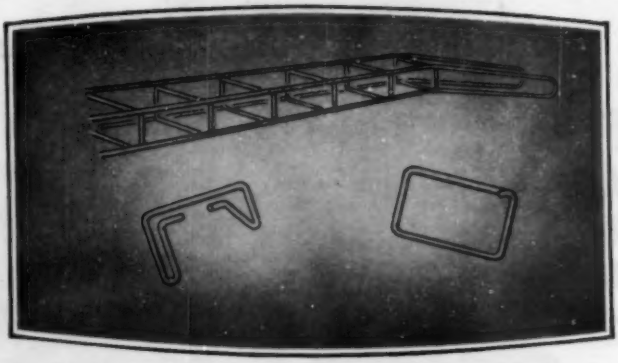
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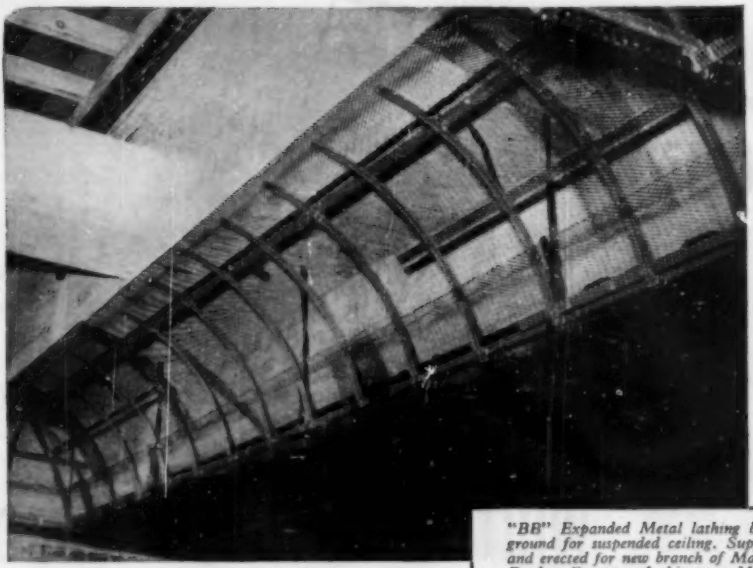
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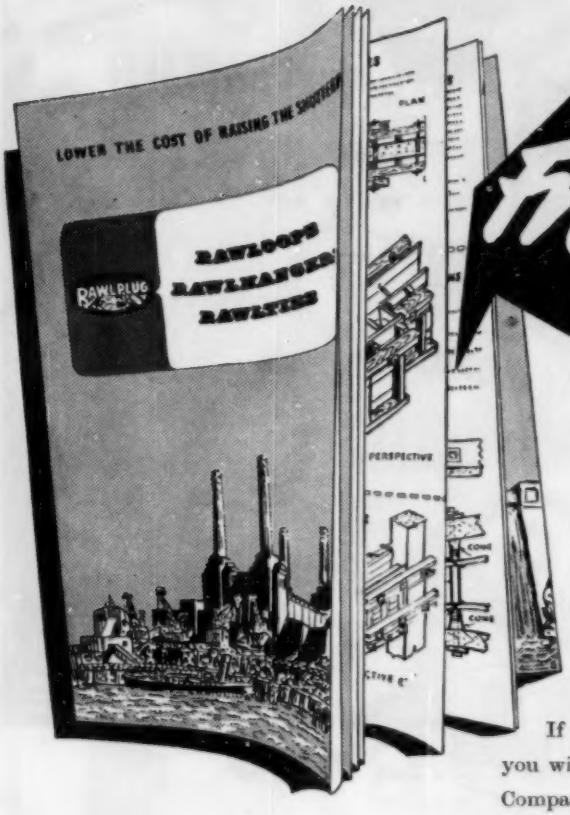
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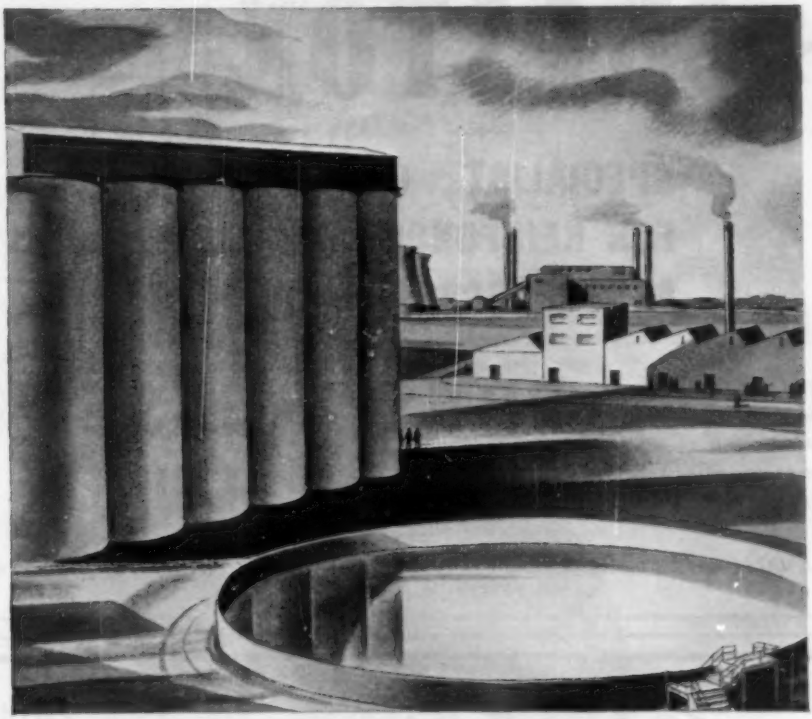
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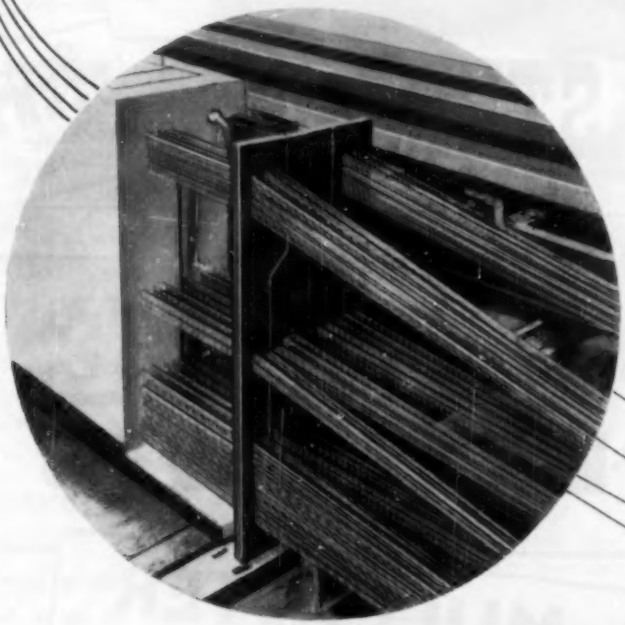
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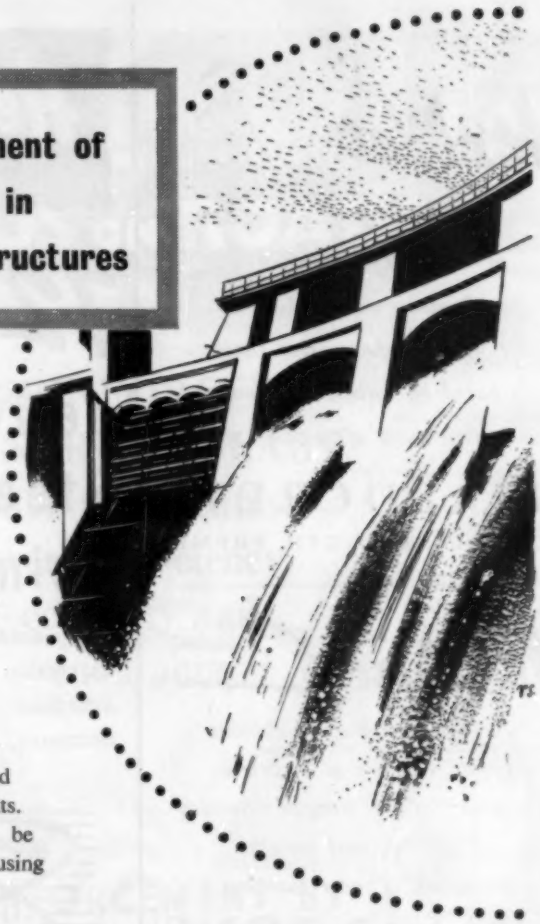
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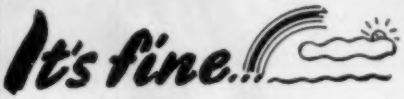
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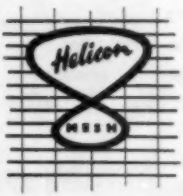
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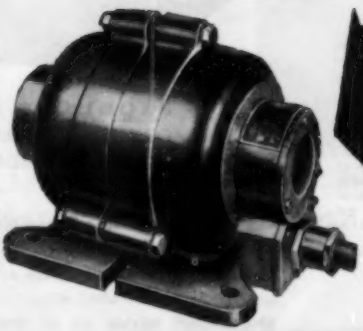
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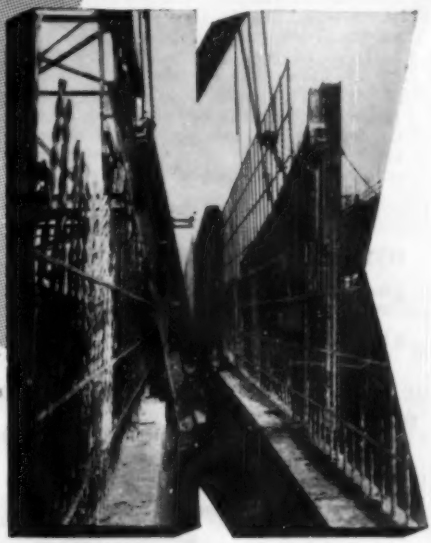
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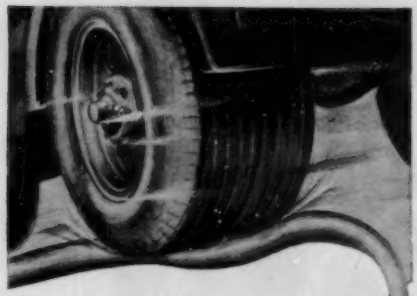
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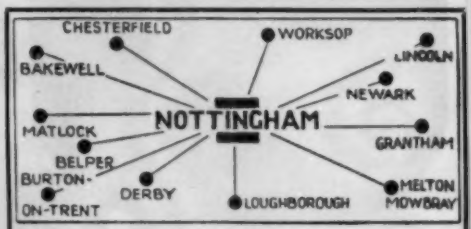
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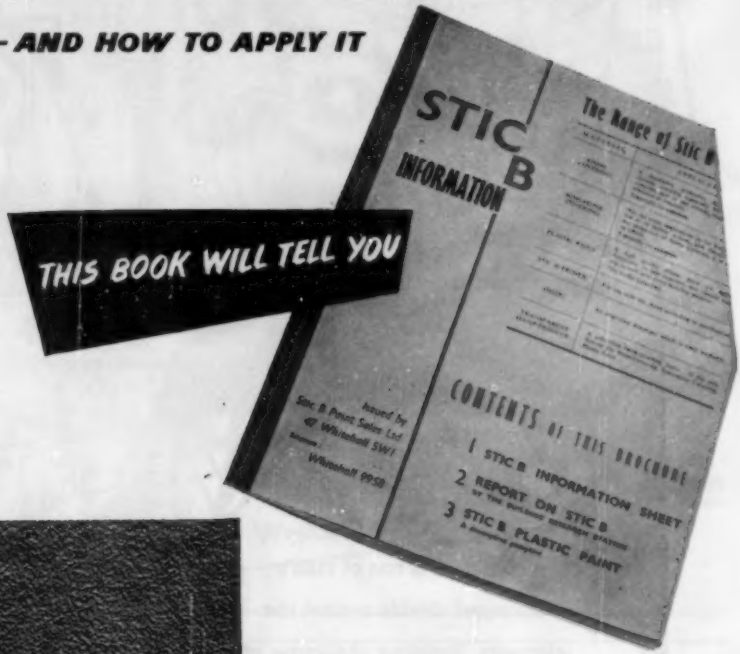
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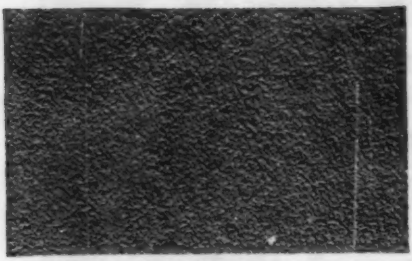
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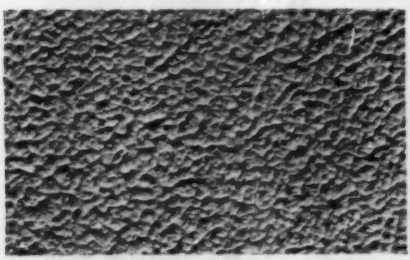
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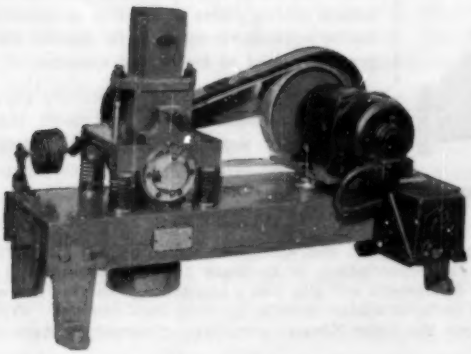
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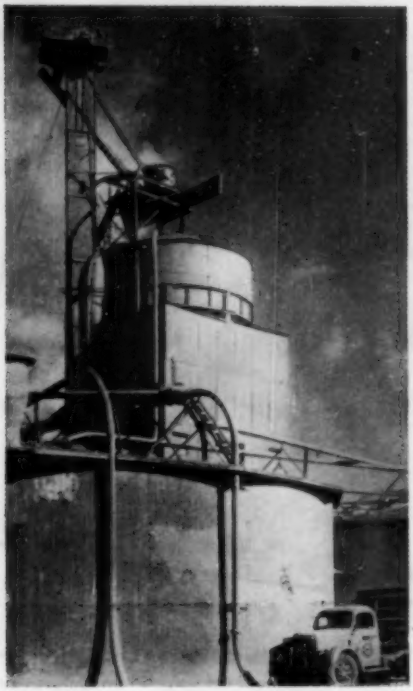


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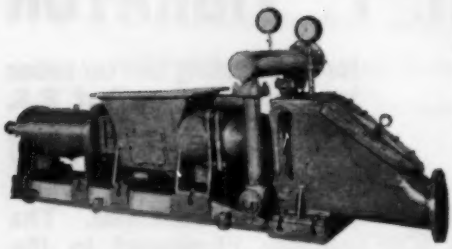


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Volume XLIX, No. 7. LONDON, JULY, 1954.

EDITORIAL NOTES

Training in Concrete Work.

THE desirability of training men employed in the making and placing of concrete has been mooted for many years, and it is gratifying to know that a start is at last being made in this direction. As was mentioned in our last number, courses of instruction in concrete work are to be held this year at several places in Great Britain. The principal aim is the training of supervisors, but the scheme also allows steel-fixers, carpenters employed on shuttering, mixer-men, and others employed on concrete work to attend the courses. The scheme has been arranged by the Reinforced Concrete Association, whose members will nominate the men to undertake the course, and who will also nominate members of their staffs to act as instructors. Diplomas will be awarded to men who pass the course satisfactorily.

In spite of the great developments that have taken place in recent years in the art of making concrete, the workers are generally unskilled labourers. It may seem anomalous that men, for example, using vibrators should be classed as common labourers when it is considered how seriously they can weaken concrete if they misuse their tools, and that men operating mixers, who can also seriously affect the strength of concrete if they fail to follow their instructions precisely, should be paid only a penny an hour more than unskilled labourers shovelling ballast or pushing barrows. It may be doubted, however, whether attendance at a series of lectures or the possession of a diploma would have much result other than a demand for higher pay. A contractor's agent on an important work told us recently how carefully he had impressed upon the men the serious ill-effects of over-vibration, but that did not prevent one of the men walking away without stopping his vibrator and returning to find it in the midst of a large pool of slurry. Conscientious work and attention to instructions are an attitude of mind resulting from environment in early youth rather than to later teaching, for men have been known to leave their duty in a signal-box and endanger lives on a railway although they knew that it was wrong to do so. It seems unfortunately true nowadays that a great deal of slackness and lack of discipline are to be expected on building sites as well as elsewhere. It also seems that the most certain way to ensure concrete of high and more uniform strength lies in more mechanisation so that quality will depend less on the desire of a man to make his work as easy as possible, or to neglect it. It is better for the employer to devise means of lightening labour rather than leave it to the men

to find their own ways of doing so. Too often work is looked upon as a necessary evil rather than a thing in which one can take a pride and from which one can derive pleasure. The best concreting is done by men regularly employed by the same contractor with whom they wish to stay, but in contracting it is often necessary to employ men sent from an employment exchange and whose main concern is to do as little as possible for as much as possible. So far as their work is concerned such men would get no more benefit from attending a course of lectures than they would get from ten-minutes' talk with their employer or a good foreman, or than could be printed on a small leaflet.

The training of supervisors is a much more important matter, for many foremen still prefer to do things as their fathers did. There has never been apprenticeship in concreting as there is in other building trades, and at most of the trades training schools, whose object is to train craftsmen, very little time is given to instruction on concrete. The carpenter or bricklayer who has become a foreman cannot be expected to have a good knowledge of concrete work, and it is to be hoped that many of them can be persuaded to take more interest in this branch of their work. Certainly it would benefit their employers and the whole industry if all foremen knew the elements of the theory of reinforced concrete and the principles of making concrete of uniformly high strength. We see little value in teaching labourers except in the case of the more intelligent among them who might make good foremen, but much advantage in having supervisors and foremen with a sound knowledge of concrete and reinforced concrete. It is possible that the best solution of the problem would be to ensure that all general foremen have a sound knowledge of the subject and at the same time to train gangers and leading hands. Foremen bricklayers, foremen carpenters, and foremen in other building trades, men with more than the usual skill and some ability to get the best work from their fellow tradesmen, have for long been recognised as useful men in the industry. There might be a similar class of foremen concretors. The cost would be less than that of attempting to make concreting a craft whose members would be paid craftsmen's rates.

An important problem is the selection of instructors. In the preliminary announcement of the scheme these men are called lecturers. This seems to be a mistake both practically and psychologically. The training of foremen and gangers is an entirely different matter from teaching young students at a university or school. Such men would not wish to be "lectured", and they would not be happy listening to young lecturers whose work has been in a laboratory and whose knowledge has been acquired from lectures and books. The best type of instructors would probably be foremen of much experience who have had a special course on concrete, who could talk to the men in words they understand and who would be respected for their knowledge and experience. They would, we hope, impart their knowledge without the use of the decimal system, and without the use of terms such as water-cement ratio, fineness modulus, and other terms which may be familiar and useful to experienced people but which might be meaningless to the men being taught. Finally, we would suggest that instead of attending classes at a school it would be better for the instructors to impart their knowledge on the construction site on which the men are working. Working men are seldom happy sitting in a classroom or watching small-scale experiments being made by a young man in a laboratory.

Analysis of Statically-indeterminate Structures by the Deformation Method.—1.

By M. SMOLIRA, Ph.D., A.M.I.C.E., D.I.C.

Introduction.

THE main purpose of this series of articles is to present time-saving methods of analysing continuous frames with curved members and Vierendeel trusses, including continuous Vierendeel trusses. This article by way of introduction deals with rapid methods of analysis of continuous beams and frames with straight members. The method described is based on the actual deformations of elastic structures and on well-known fundamental relations between the stresses and strains, and requires the setting out and solving of simultaneous equations of equilibrium of angular deformations of the structure by expressing the conditions of compatibility of deformations of the members.

The method is " exact ", that is it includes no more assumptions or limitations than those inherent in the elastic theory, and it provides a rapid solution of many problems which often present mathematical difficulties seemingly out of proportion to their importance. A wide range of problems concerned with elastic structures may successfully be solved by the method. Many of the equations given are applicable in a general case for unsymmetrical frames of any shape, with prismatic or non-prismatic members, and with any system of loading.

The method is not " automatic ". It is particularly necessary to visualize the deformed shape of the structure, and this should be drawn with some exaggeration in order to set out properly the equations of equilibrium. Every figure and every operation has its meaning and its clear physical interpretation, thus helping the setting out of equations of equilibrium and avoiding errors. It should also be remembered that a correct solution of simultaneous equations is not proof of a correct result unless the equations are properly set out. No sign convention is necessary, moments and forces being assumed to act in the directions necessary to close the angular or linear gaps caused by the relaxation of the continuity of joints. The degree of indeterminacy is apparent by inspection in every case, and no formulæ for this are given or are necessary. Sway is taken into account in the actual deformations of the structures, and therefore no separate treatment of sway is required. Little extra effort is required to take into account beams or frames of different spans and cross sections, or with non-prismatic members. Deflections, although generally eliminated from the equations at early stages, can be easily calculated, as will be shown in examples.

Although many equations and formulæ are derived, it is not necessary to remember any of them or to refer to any set of equations given. In each case equations can be written immediately by simple reasoning, with the help of load functions and elastic constants for each beam or column. It is not proposed to supply ready-made formulæ for various cases, and no effort has been made to present or tabulate them. However, in the course of the analyses, some solutions are given in explicit form, and these may be compared with the results in books on the theory of structures. Numerical examples are given to illustrate the application of the theory.

Notation.

A, B, C, etc., Joints.
A, Area of $\frac{m}{EI}$ diagram.
a, b, Position of a concentrated load.
c, Coefficients.
d, Depth of beam.
E, Modulus of elasticity.
h, Height of columns.
H, Horizontal forces in frames.
I, Moments of inertia.
k, Coefficients.
L, Spans.
m, Bending moments.
n, Modular ratio: $n = \frac{E_s}{E_c}$; also $n = \frac{I_1}{I_2}$.
p, The rise of curved beams; also pressure.
P, Forces.
R, Radius of circular beams.
s, Length of inclined beams.
S, Forces in struts.
S.c., Shearing condition.
V, Shearing forces.
w, Uniformly-distributed load.
W, Concentrated forces.
x, y, Co-ordinates.
$\bar{x}$, $\bar{z}$, Position of centre of gravity of $\frac{m}{EI}$ diagram.
α, β, Angular deformations of beam due to unit bending moment.
γ, δ, Angular deformations of beam due to unit horizontal force.
Δ, Deflections of beams and frames; Horizontal sway in frames and trusses.
θ, Load functions.
ϕ, Half central angle of circular beams.

I.—CONTINUOUS BEAMS.

General Case.—A continuous beam of any shape, and subjected to the action of any system of loading and settlement of the supports, may be conveniently analysed by considering the angular deformations of the beam. Statically-indeterminate bending moments can be found by setting out equations of equilibrium of angular deformations of the joints.

First assume that the beam is cut at the supports and thus relaxed from the effect of continuity, and calculate angular deformations θ of each beam due to the external system of loading (*Fig.* 1). The angular gaps at each support must now be closed by applying statically-indeterminate bending moments of such values that all gaps are closed at the same time. For example, at support B (*Fig.* 1*b* and 1*c*) the condition of equilibrium is

$$\left.\begin{aligned} &\sum \rho_b = \sum \theta_b \pm \sum \frac{\Delta}{L} \quad . \quad . \quad . \quad . \\ \text{or} \quad &m_a\beta_{ba} + m_b\alpha_{ba} + m_b\alpha_{bc} + m_c\beta_{bc} = (\theta_{ba} + \theta_{bc}) \pm \left(\frac{\Delta}{L_1} + \frac{\Delta}{L_2}\right) \quad . \end{aligned}\right\} \quad (1)$$

in which ρ represents rotations of the beam due to statically-indeterminate

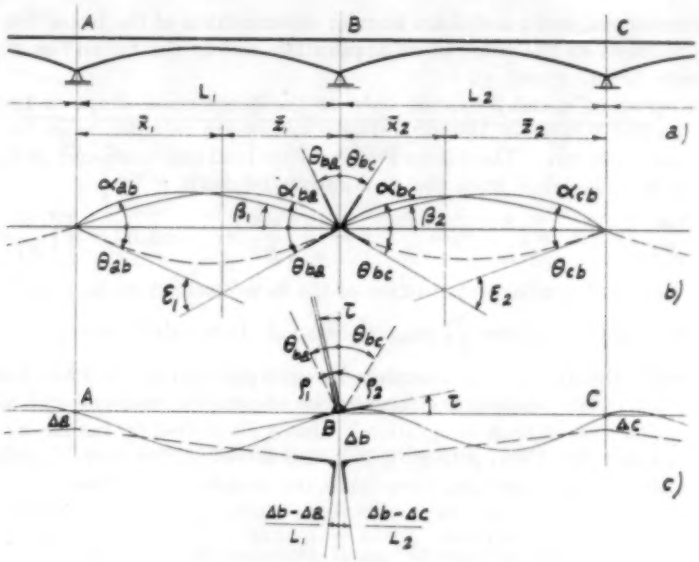


Fig. 1.

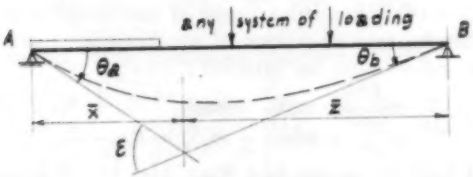


Fig. 2.

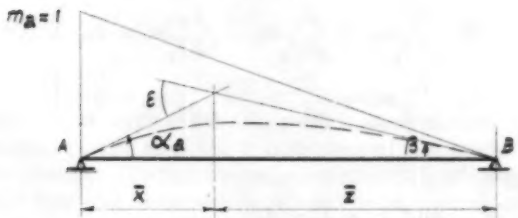


Fig. 3.

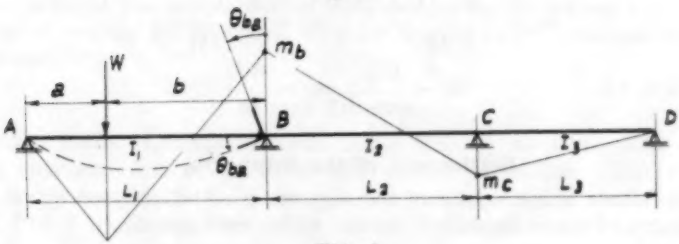


Fig. 4.

bending moments, and α and β are angular deformations of the beam due to unit bending moment at the point of its application and at the far end of the beam respectively (*Figs.* 2 and 3).

The values of α and β depend only on the geometrical shape of the beam, and not on the system of loading. These values are referred to as the elastic constants of the beam. The values θ (*Fig.* 2) are load functions and, in a general case, may be calculated from the well-known relations

$$\varepsilon = \int_A^B \frac{m}{EI}.dx\,;\ \theta_a = \varepsilon\frac{\bar{z}}{L}\,;\ \theta_b = \frac{\bar{x}}{L}\,;\ \bar{x} = \frac{1}{\varepsilon}\int_A^B \frac{mx}{EI}.dx\,;\ \text{and}\ \Delta_x = \iint \frac{m}{EI}dx\,. \quad (2)$$

in which ε is total angular deformation of the beam from A to B, $\bar{x}$ and $\bar{z}$ define the centre of gravity of the $\frac{m}{EI}$ diagram, and Δ_x is the deflection at any point.

Generally the formulæ are calculated from equations (2), but for more complicated loading or non-prismatic beams the summation method will probably lead to a quicker result than integration. This is illustrated by various examples. The elastic constants α and β (*Fig.* 3) can also be calculated from equations (2). For prismatic beams, however, they have the well-known values

$$EI\varepsilon = \frac{L}{2},\ EI\alpha = \frac{L}{3},\ EI\beta = \frac{L}{6} \quad . \quad . \quad . \quad . \quad (3)$$

EXAMPLES.—The following examples illustrate the procedure.

In a general case of a three-span beam of any shape and with any system of loading, the following two equations of equilibrium of angular deformations at the supports B and C must be satisfied:

$$\left.\begin{aligned} \text{B},\ & m_b\alpha_{ba} + m_b\alpha_{bc} + m_c\beta_2 = \theta_{ba} + \theta_{bc} \\ \text{C},\ & m_c\alpha_{cb} + m_c\alpha_{cd} + m_b\beta_2 = \theta_{cb} + \theta_{cd} \end{aligned}\right\} \quad . \quad . \quad . \quad (4)$$

For a concentrated load W on the first span, and for prismatic beams (*Fig.* 4), these equations reduce to

$$\left.\begin{aligned} \text{B},\ & \frac{L_1}{3EI_1}m_b + \frac{L_2}{3EI_2}m_b - \frac{L_2}{6EI_2}m_c = \theta_{ba} \\ \text{C},\ & \frac{L_3}{3EI_3}m_c + \frac{L_2}{3EI_2}m_c - \frac{L_2}{6EI_2}m_b = 0 \end{aligned}\right\} \quad . \quad . \quad . \quad (5)$$

in which $\theta_{ba} = \frac{m_o}{6EI_1}(L_1 + a)$.

For $L_1 = L_2 = L_3$ and $I_1 = I_2 = I_3$:

$$m_b = \frac{4Wab(L+a)}{15L^2}\,;\ m_c = \frac{Wab(L+a)}{15L^2} \quad . \quad . \quad . \quad (6)$$

For $a = b = \frac{L}{2}$: $\qquad m_b = \frac{WL}{10}\,;\ m_c = \frac{WL}{40} \quad . \quad . \quad . \quad . \quad (7)$

Settlement of the Supports.

The influence of settlement of the supports may be analysed by the use of equations (1). This is illustrated by the following examples.

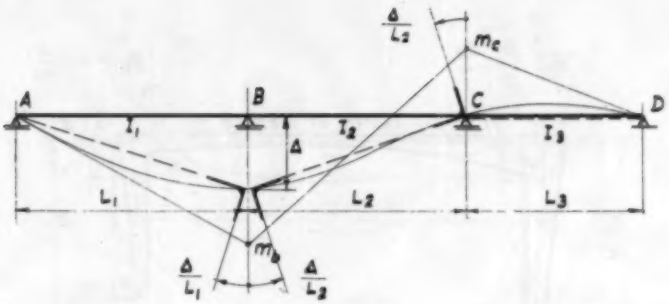


Fig. 5.

THREE-SPAN BEAM SUBJECTED TO SETTLEMENT OF THE INTERMEDIATE SUPPORT (*Fig.* 5).—Denoting by Δ the settlement of support B, the angular gaps at B and C in the beam relaxed from the effect of continuity are $\left(\frac{\Delta}{L_1}+\frac{\Delta}{L_2}\right)$ and $\frac{\Delta}{L_2}$ respectively. In a general case of non-prismatic beams the following equations of equilibrium for supports B and C may be set out:

$$\text{(B) } m_b\alpha_{ba}+m_b\alpha_{bc}-m_c\beta_{bc}=\frac{\Delta}{L_1}+\frac{\Delta}{L_2};\ \text{(C) } m_c\alpha_{cb}+m_c\alpha_{cd}-m_b\beta_{cb}=\frac{\Delta}{L_2}. \quad (8)$$

For prismatic beams these equations reduce to

$$\left.\begin{aligned}&\text{B, } \frac{L_1}{3EI_1}m_b+\frac{L_2}{3EI_2}m_b-\frac{L_2}{6EI_2}m_c=\frac{\Delta}{L_1}+\frac{\Delta}{L_2}\\&\text{C, } \frac{L_2}{3EI_2}m_c+\frac{L_3}{3EI_3}m_c-\frac{L_2}{6EI_2}m_d=\frac{\Delta}{L_2}\end{aligned}\right\} \quad . \quad . \quad (9)$$

For $L_1=L_2=L_3$ and $I_1=I_2=I_3$: $m_b=\frac{18\Delta EI}{5L^2}$; $m_c=\frac{12\Delta EI}{5L^2}$. (10)

II.—FRAMES.

Elastic frames can be analysed by the deformation method in a similar way to continuous beams, except that the translations of the joints, or sway, must also be taken into account. To find the values of statically-indeterminate bending moments, consider that the structure is relaxed from continuity at the joints, apply the external system of loading, and calculate angular gaps at the joints, taking into account the translations of the joints. Finally, set out equations of equilibrium of angular deformations, with the help of elastic constants of beams and columns.

Portal Frames.

GENERAL CASE.—Consider a portal frame (*Fig.* 6), with prismatic or non-prismatic members, and subjected to any system of loading. The deflected shape of the frame, in the condition relaxed from continuity at the joints, is shown in *Fig.* 6 by dotted lines. The values of Δ represent a final horizontal

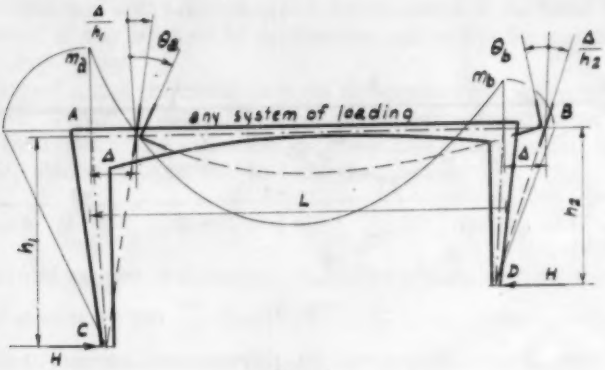


Fig. 6.

translation of the joints. The angular gaps at A and B are $\left(\theta_a - \frac{\Delta}{h_1}\right)$ and $\left(\theta_b + \frac{\Delta}{h_2}\right)$ respectively, θ representing load functions as defined by equation (2). These angular gaps must now be closed by the bending moments m_a and m_b, which, together with the sway Δ, are calculated from two conditions of equilibrium of the joints A and B, and from one condition of shear. These are as follows:

$$\left.\begin{aligned} &\text{CAB, } m_a\alpha_{ab} + m_a\alpha_{ac} + m_b\beta = \theta_a - \frac{\Delta}{h_1} \\ &\text{ABD, } m_b\alpha_{ba} + m_b\alpha_{bd} + m_a\beta = \theta_b + \frac{\Delta}{h_2} \\ &\text{S.c., } \frac{m_a}{h_1} = \frac{m_b}{h_2} \end{aligned}\right\} \quad . \quad . \quad . \quad (11)$$

in which S.c. is the "shearing condition".

For prismatic beams and columns, with any system of loading, these equations reduce to

$$\left.\begin{aligned} &\text{CAB, } m_a\frac{L}{3EI_b} + m_a\frac{h_1}{3EI_1} + m_b\frac{L}{6EI_b} = \theta_a - \frac{\Delta}{h_1} \\ &\text{ABD, } m_b\frac{L}{3EI_b} + m_b\frac{h_2}{3EI_2} + m_a\frac{L}{6EI_b} = \theta_b + \frac{\Delta}{h_2} \\ &\text{S.c., } \frac{m_a}{h_1} = \frac{m_b}{h_2} \end{aligned}\right\} \quad . \quad . \quad (12)$$

Portal Frame with Uniformly-distributed Load on Part of Beam (*Fig.* 7).—The load functions θ_a and θ_b, for a beam with uniformly-distributed load on part of its length, are $EI_b\theta_a = \frac{wa^2}{24L}(2L - a)^2$ and $EI_b\theta_b = \frac{wa^2}{24L}(2L^2 - a^2)$. Substituting these values in (12),

$$m_a = \frac{wa^2h_1}{8L} \cdot \frac{h_1(2L - a)^2 + h_2(2L^2 - a^2)}{(n_1h_1^3 + n_2h_2^3) + L(h_1^2 + h_1h_2 + h_2^2)} \quad . \quad . \quad (13)$$

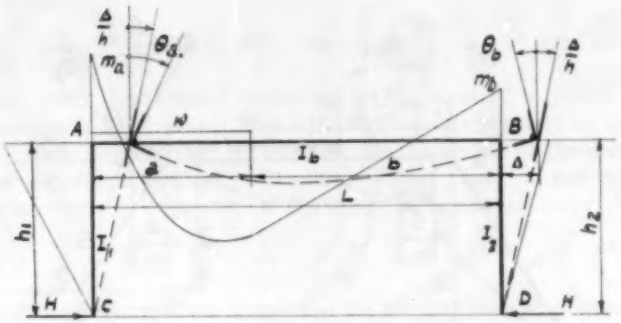


Fig. 7.

$$m_b = \frac{h_2}{h_1} m_a \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (14)$$

in which $n_1 = \frac{I_b}{I_{c1}}$, $n_2 = \frac{I_b}{I_{c2}}$, and, if $h_1 = h_2 = h$ and $I_1 = I_2 = I_c$, and $n = \frac{I_b}{I_c}$,

$$m_a = m_b = \frac{wa^2}{4} \cdot \frac{3L - 2a}{2nh + 3L} \quad . \quad . \quad . \quad . \quad (15)$$

PORTAL FRAME WITH A CRANE LOAD.—If one column is eccentrically loaded the deflected shape of the frame in a relaxed condition is as indicated in *Fig.* 8. The load function θ_{ac} is $EI\theta_{ac} = \frac{We}{6h^2}(a^3 + 3a^2.b - 2b^3)$, and (12) becomes

$$\left.\begin{aligned} &\text{A, } m_a\left(\frac{L}{3EI_b} + \frac{h_1}{6EI_1}\right) - m_b\frac{L}{6EI_b} = \frac{\Delta}{h} - \frac{We}{6h^2EI_b}(a^3 + 3a^2b - 2b^3) \\ &\text{B, } m_b\left(\frac{L}{3EI_b} + \frac{h}{3EI_2}\right) - m_a\frac{L}{6EI_b} = \frac{\Delta}{h} \\ &\text{S.c., } \frac{m_o}{h} - \frac{m_a}{h} = \frac{m_b}{h} \end{aligned}\right\} \quad . \quad (16)$$

in which $m_o = We$. For $I_b = I_1 = I_2$,

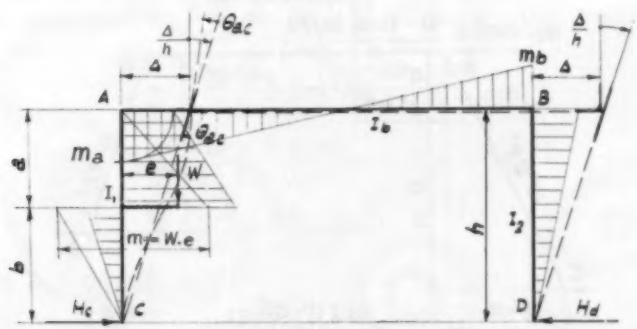


Fig. 8.

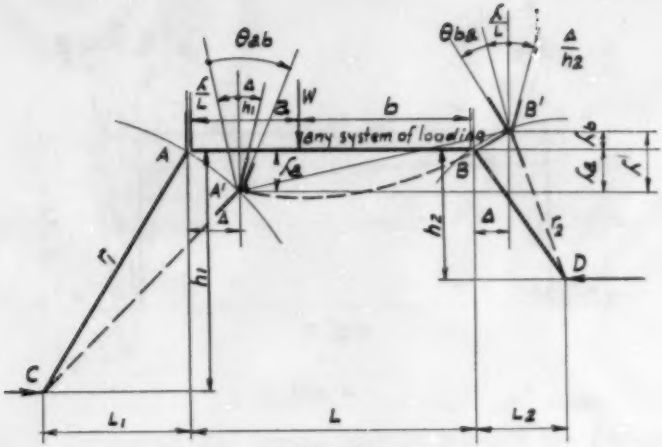


Fig. 9.

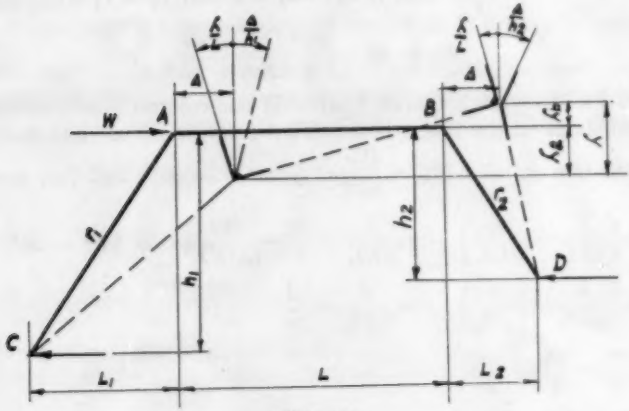


Fig. 10.

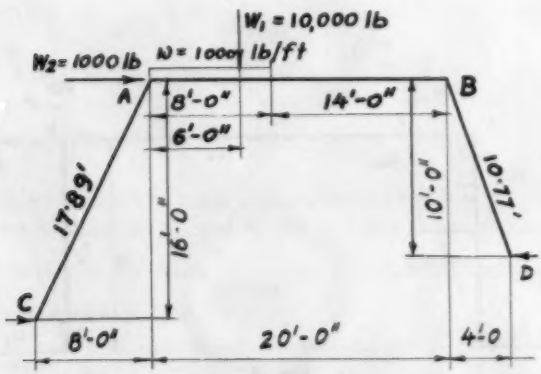


Fig. 11.

$$m_a = \frac{We}{2h(3L+2h)}(h^2+3hL+9b^2)\,;\quad m_b = \frac{3We}{2h(3L+2h)}(h^2+hL-3b^2)\ .\quad (17)$$

FRAMES WITH INCLINED COLUMNS.—In the case of a portal frame with inclined columns (*Fig.* 9), points A and B will move horizontally by Δ, and vertically by λ_a and λ_b respectively. Neglecting the shortening of the columns due to the axial loads, these vertical translations of the joints do not represent new unknowns, and can be expressed in terms of the horizontal sway Δ as follows:

$$\lambda_a = \Delta\frac{L_1}{h_1}\,;\quad \lambda_b = \Delta\frac{L_2}{h_2}\,;\quad \lambda = \lambda_a + \lambda_b.$$

The deformed shape of the frame and the angular gaps at A and B are shown in *Fig.* 9. The equations of equilibrium can now be set out, for any system of loading and any shape of frame, as follows:

$$\left.\begin{aligned} &\text{A},\ m_a\alpha_{ab} + m_a\alpha_{ac} + m_b\beta_{ab} = \theta_{ab} - \frac{\Delta}{h_1} - \frac{\lambda}{L}\\ &\text{B},\ m_b\alpha_{ba} + m_b\alpha_{bd} + m_a\beta_{ba} = \theta_{ba} + \frac{\Delta}{h_2} + \frac{\lambda}{L}\\ &\text{S.c.},\ \frac{m_a}{h_1} = \frac{m_b}{h_2}\end{aligned}\right\}\quad .\quad .\quad (18)$$

FRAME WITH INCLINED COLUMNS SUBMITTED TO THE ACTION OF A LATERAL FORCE.—The deformed frame is as shown in *Fig.* 10, and the equations of equilibrium are

$$\left.\begin{aligned} &\text{A},\ m_a\alpha_{ab} + m_a\alpha_{ac} - m_b\beta_{ab} = \frac{\Delta}{h_1} + \frac{\lambda}{L}\\ &\text{B},\ m_b\alpha_{ba} + m_b\alpha_{bd} - m_a\beta_{ba} = \frac{\Delta}{h_2} + \frac{\lambda}{L}\\ &\text{S.c.},\ \frac{m_a}{h_1} + \frac{m_b}{h_2} = W\end{aligned}\right\}\quad .\quad .\quad .\quad (19)$$

in which the value of λ is as defined in the previous section.

EXAMPLE.—Consider the frame in *Fig.* 11. To make the analysis clearer, the influence of each load will be dealt with separately.

For a point load $W_1 = 10{,}000$ lb. at a distance $a = 6$ ft. from A, the load functions are:

$$\theta_a = \frac{10\times 6\times 14}{6\times 20}(20+14) = 238\,;$$

$$\theta_b = \frac{10\times 6\times 14}{6\times 20}(20+6) = 182\,;$$

$$\lambda = \frac{8\Delta}{16} + \frac{4\Delta}{10} = 0{\cdot}9\Delta.$$

From Equation 18

$$\text{A},\ m_a\frac{20}{3} + m_a\frac{17{\cdot}89}{3} + m_b\frac{20}{6} = 238 - \frac{\Delta}{16} - \frac{0{\cdot}9\Delta}{20}\,;$$

$$\text{B, } m_b\frac{20}{3} + m_b\frac{10{\cdot}77}{3} + m_a\frac{20}{6} = 182 + \frac{\Delta}{10} + \frac{0{\cdot}9\Delta}{20};$$

$$\text{S.c., } \frac{m_a}{16} = \frac{m_b}{10},$$

from which $m_a = 17{,}000$ ft.-lb. and $m_b = 10{,}620$ ft.-lb. It is interesting to note that, if the columns were vertical, from equation (12)

$$m_a = \frac{10 \times 6 \times 14}{2 \times 20} \cdot \frac{16 + 16(20 + 14) + 10(20 + 6)}{16^3 + 10^3 + 20(16^2 + 10 \times 16 + 10^2)} = 17{,}520 \text{ ft.-lb.}$$

$$m_b = 17{\cdot}52 \times \frac{10}{16} = 10{,}950 \text{ ft.-lb.}$$

and that if both the columns were of the same height ($h_1 = h_2 = 13$ ft.),

$$m_a = m_b = \frac{3 \times 6 \times 14 \times 10}{2(2 \times 13 + 3 \times 20)} = 14{,}650 \text{ ft.-lb.}$$

For a horizontal force $W_2 = 1000$ lb. acting at A (Fig. 11), equations (19) become

$$\text{A, } m_a\frac{17{\cdot}89}{3} + m_a\frac{20}{3} - m_b\frac{20}{6} = \frac{\Delta}{16} + \frac{0{\cdot}9\Delta}{20};$$

$$\text{B, } m_b\frac{10{\cdot}77}{3} + m_b\frac{20}{3} - m_a\frac{20}{6} = \frac{\Delta}{10} + \frac{0{\cdot}9\Delta}{20};$$

$$\text{S.c., } \frac{m_a}{16} + \frac{m_b}{10} = 1{,}000,$$

from which $m_a = 4980$ ft.-lb. and $m_b = 6880$ ft.-lb.

For a uniformly-distributed load on part of the beam (*Fig.* 11), the load functions are

$$\theta_{ab} = \frac{1 \times 8^2}{24 \times 20}(2 \times 20 - 8)^2 = 136{\cdot}5 \text{ and } \theta_{ba} = \frac{1 \times 8}{24 \times 20}(2 \times 20^2 - 8^2) = 98{\cdot}1,$$

and from (18)

$$\text{A, } m_a\frac{20}{3} + m_a\frac{17{\cdot}89}{3} + m_b\frac{20}{6} = 136{\cdot}5 - \frac{\Delta}{16} - \frac{0{\cdot}9\Delta}{20};$$

$$\text{B, } m_b\frac{20}{3} + m_b\frac{10{\cdot}77}{3} + m_a\frac{20}{6} = 98{\cdot}1 + \frac{\Delta}{10} + \frac{0{\cdot}9\Delta}{20};$$

$$\text{S.c., } \frac{m_a}{16} = \frac{m_b}{10},$$

from which $m_a = 9540$ ft.-lb. and $m_b = 5960$ ft.-lb.

Continuous Frames with Straight Members.

GENERAL CASE.—Continuous frames may be analysed in a similar way to single-span frames. An increased number of unknowns is accompanied by a similar increase in the number of conditions of equilibrium.

In an imaginary condition of the frame, with joints relaxed from the effect

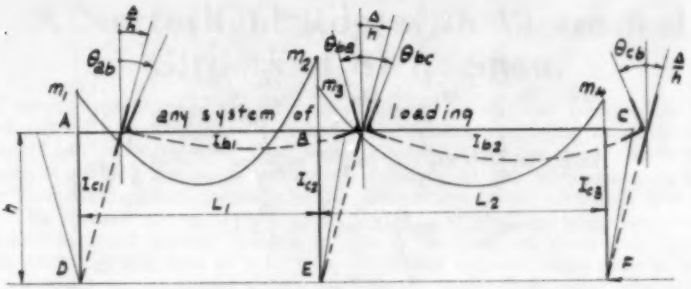


Fig. 12.

of continuity, every external joint with two members, as A or C in *Fig.* 12, provides one condition of equilibrium, while an internal joint (as B) with three members provides two conditions. The intermediate junction, such as B in *Fig.* 12, therefore has two angular gaps, that is between members ABC and ABE (or alternatively EBC). When all angular gaps are marked, taking sway into account, the equations of equilibrium can be set out in the usual way. This procedure is illustrated in the following examples.

For a two-span frame (*Fig.* 12), with any shape of members and any system of loading, the equations of equilibrium are

$$\left.\begin{aligned}
&\text{DAB},\ m_1\alpha_{ab} + m_1\alpha_{ad} + m_2\beta_{ab} = \theta_{ab} - \frac{\Delta}{h}\\
&\text{ABC},\ m_1\beta_{ba} + m_2\alpha_{ba} + m_3\alpha_{bc} + m_4\beta_{bc} = \theta_{ba} + \theta_{bc}\\
&\text{ABE},\ m_1\beta_{ba} + m_2\alpha_{ba} + (m_2 - m_3)\alpha_{be} = \theta_{ba} + \frac{\Delta}{h}\\
&\text{BCF},\ m_3\beta_{cb} + m_4\alpha_{cb} + m_4\alpha_{cf} = \theta_{cb} + \frac{\Delta}{h}\\
&\text{S.C.},\ \frac{m_1}{h} = \frac{m_2 - m_3}{h} + \frac{m_4}{h}
\end{aligned}\right\} \quad . \quad . \quad (20)$$

When bending moments are calculated from (20) all the other values can be obtained from equations of statics.

Two-span Frame with Uniformly-distributed Load on One Beam (*Fig.* 13).—If both beams are of equal span and the frame has constant moments of inertia, equations (20) reduce to

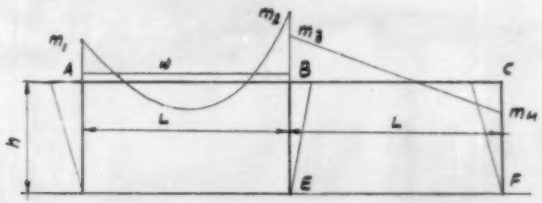


Fig. 13.

$$\left.\begin{aligned}
&\text{DAB, } m_1\frac{L}{3}+m_1\frac{h}{3}+m_2\frac{L}{6}=\frac{wL^3}{24}-\frac{\Delta}{h}EI\\
&\text{ABC, } m_1\frac{L}{6}+m_2\frac{L}{3}+m_3\frac{L}{3}-m_4\frac{L}{6}=\frac{wL^3}{24}\\
&\text{ABE, } m_1\frac{L}{6}+m_2\frac{L}{3}+(m_2-m_3)\frac{h}{3}=\frac{wL^3}{24}+\frac{\Delta}{h}EI\\
&\text{BCF, } -m_3\frac{L}{6}+m_4\frac{L}{3}+m_4\frac{h}{3}=\frac{\Delta}{h}EI\\
&\text{S.c., } m_1=m_2-m_3+m_4.
\end{aligned}\right\} \quad . \quad . \quad (21)$$

The general solution of these equations gives

$$\left.\begin{aligned}
m_1&=\frac{wL^3}{24}\cdot\frac{6L+7h}{3L^2+7hL+4h^2}. & m_2&=\frac{wL^2}{24}\cdot\frac{6L^2+13hL+6h^2}{3L^2+7hL+4h^2}\\
m_3&=\frac{whL^2}{24}\cdot\frac{6h+5L}{3L^2+7hL+4h^2}. & m&=\frac{whL^3}{24}\cdot\frac{1}{3L^2+7hL+4h^2}
\end{aligned}\right\} \cdot \quad (22)$$

(To be continued.)

A Circular Diagram for the Design of Beams.

On page 160 of our number for May, 1954, in line one of column one, for *Fig.* 1 read *Fig.* 3; in line one of column two, for *Fig.* 2 read *Fig.* 4; and in line eighteen, for *Fig.* 1 read *Fig.* 4.

A Steel Jig for Positioning Piles.

The piles in the causeway between Port Isabel and Padre Island, Texas, U.S.A., have been driven with the aid of a movable steel jig (*Fig.* 1). The jig consisted of two trusses, the tension members of which were made with wire ropes, and two platforms one above the other on which were fixed the pile-driving guides. The trusses were supported at their centres on the group of piles last driven and one end was anchored to the penultimate group. On each of the two platforms at the other end were four guide-frames, one for each pile, with one hinged side which was opened to admit the pile and closed and secured before the piles were driven. The jig was about 62 ft. long, weighed about 17 tons, and accommodated the two vertical and two raking piles forming a group. After the four piles had been driven the jig was lifted and moved forward by a crane mounted on a barge. The foregoing is from " Engineering News-Record " for April 29, 1954.

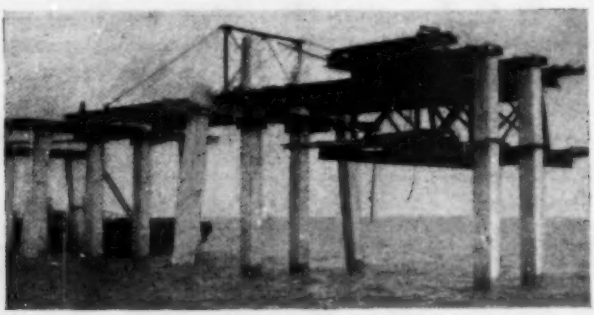


Fig. 1.

A North-light Roof with Vierendeel Girders of 81 ft. Span.

DURING recent extensions of the premises in Stockport of the Cheshire Sterilized Milk Co., Ltd., a north-light roof was constructed of which the main members are four Vierendeel girders spanning 80 ft. 10 in. The girders are at 13 ft. 2 in. centres and support precast rafters, inclined at about 45 deg. and at 2 ft. 3 in. centres, on which are woodwool slabs 2 in. thick, a 1-in. layer of sand and cement, and bituminous felt. *Fig.* 1 shows the roof from inside the building and *Fig.* 2 gives the main dimensions of a girder. It is believed that these girders are the largest of their type in reinforced concrete yet constructed in this country, although Vierendeel girders of longer spans and generally with curved top booms have been used for bridges notably in Belgium and the Belgian Congo.

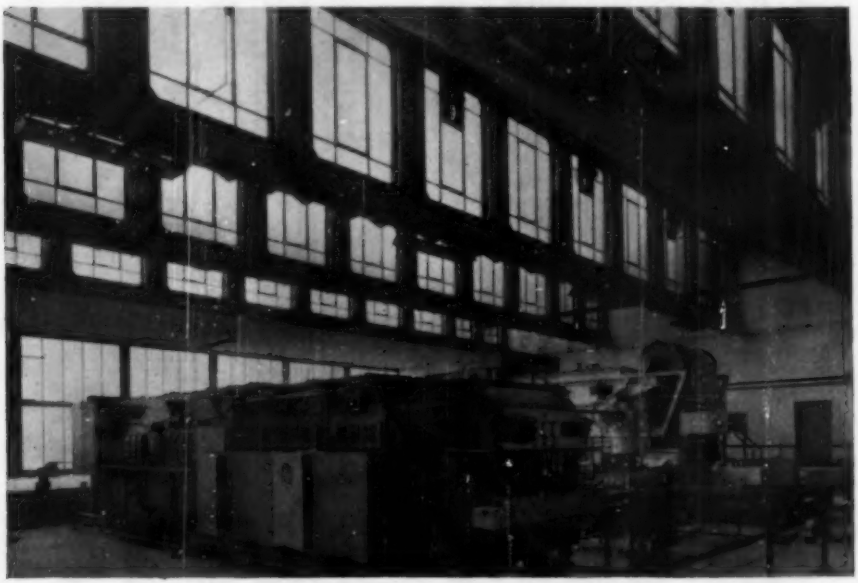


Fig. 1.—The Completed Girders during the Installation of the Plant.

Design of the Girders.

In the usual form of trussed girder with sloping and vertical web members the direct forces are transmitted between the booms mainly by direct forces in the sloping members, and the strength in bending of the web members is only a secondary factor although in a structure with rigid joints an important one. With a Vierendeel, or open-web, girder with vertical web-members only in which the direct forces are small, the stability of the structure depends primarily on the resistance to bending and shearing of these members and their connections to the booms.

Except for girders of small span and with few panels a Vierendeel girder is many times statically indeterminate and the calculation of bending moments requires the use of a method of converging approximations if the solution of a large number of simultaneous equations is to be avoided. In this case the method adopted was first to calculate the bending moments assuming that each member forming a panel of a girder had a central

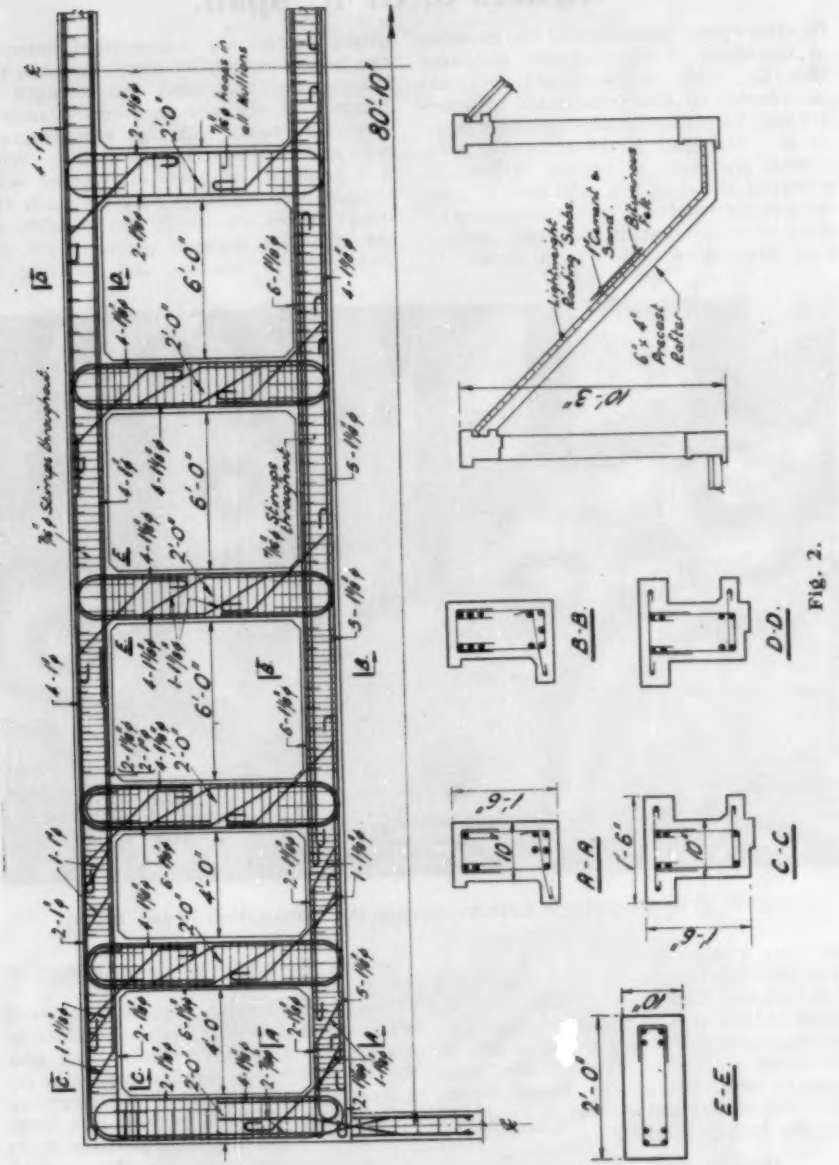

80'-10"
Light-weight Roofing Slabs
Cement & Sand
Bituminous Felt
6"x 4" Precast Rafter
10'-3"
B-B
D-D.
A-A
C-C
E-E
2'-0"

Fig. 2.

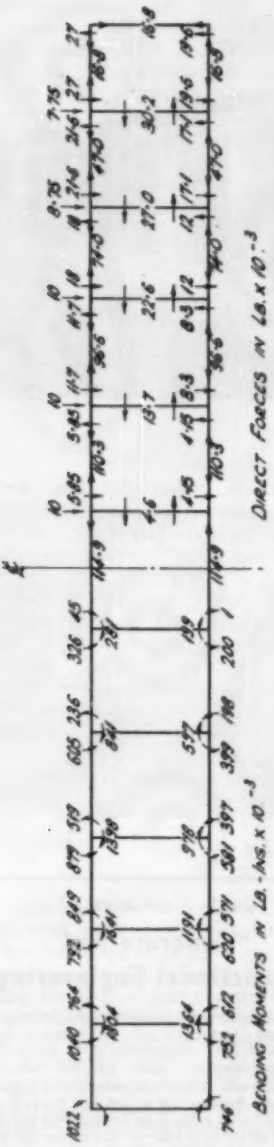


Fig. 3.

point of contraflexure. These moments were then corrected for the effect of the continuity of each panel taken separately and for the effect on a panel of the bending moments on the members of adjacent panels. No more than three corrections were required for any case, as the fourth correction would have been negligible. From the bending moments thus obtained the direct forces and shearing forces were calculated ; the values of these forces and bending moments are given in *Fig.* 3. In these calculations the stiffnesses of the members were computed from the moments of inertia of the concrete and the steel (taking account of the modular ratio) for the upper boom and web members, and for the steel only for the lower boom as the latter would be subjected to a large direct tensile force which it was considered would crack the concrete. The areas of steel used for calculating the moments of inertia were based upon a preliminary approximate analysis.

The reinforcement in a girder is shown in *Fig.* 2, which also gives a section through part of the roof showing the arrangement of the rafters. Lateral stability of the top booms and resistance to transverse wind forces were provided by reinforced concrete braces 1 ft. by 10 in. in cross section connecting the top booms of adjacent girders at their ends and at about one-third points of the span.

The effect on a girder of stresses due to shrinking of the concrete on setting and to thermal movements was considered to be negligible, as the girders are supported on columns, 1 ft. 1½ in. by 10½ in. by 14 ft. high, which are flexible compared with a girder. Nevertheless the bars in a column were not carried directly into a girder but were stopped at the soffit of the lower booms, and the connection between a column and a girder was made by crossed bars forming a hinge.

Construction.

The girders were cast in timber shutters supported on tubular scaffold. As the concreting was carried on continuously until a girder was completed it was necessary to provide shutters to the upper face of the lower boom, as the pressure of concrete in the vertical members and the top boom would otherwise have caused the concrete to flow out of the lower boom.

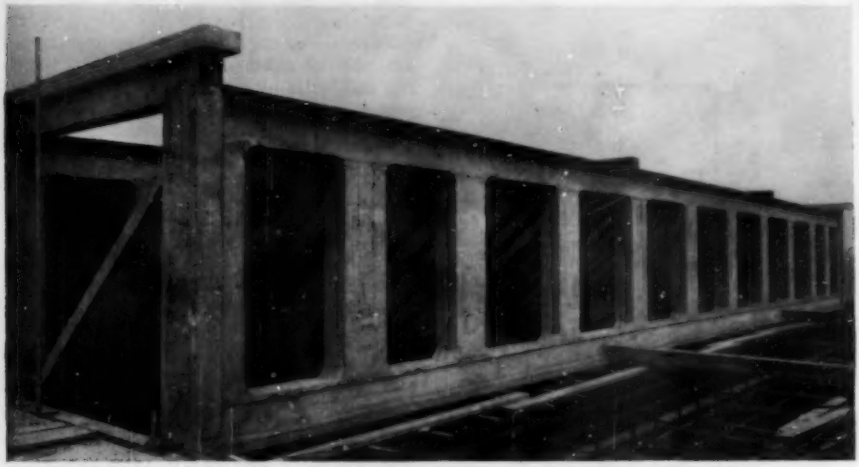


Fig. 4.

About 6½ hours were required for the concreting of a girder. The concreting was done by eight men, four filling the shutters and four using vibrators. To ensure that the concrete in the lower part of a girder was properly compacted, inspection doors about 9 in. square were formed on one face of the lower part of the shutters of each vertical member. *Fig.* 4 shows a girder after the shutters were removed and the rafters placed.

The reinforcement was delivered bent to the site; the steel for the booms was assembled in the shutters, but the bars in the vertical members were formed into cages before being lifted into position. The bars in the bottom boom were butt-welded on a platform adjacent to the shutters. The positions of the welds were staggered. After every fourth weld incorporated in the structure a sample weld was made for testing in tension. One bar more than was required for each girder was welded and a bar was chosen arbitrarily from which parts including the welds were cut. These parts were subjected to tensile tests and in all cases failure occurred in the original bar rather than in the weld. The cover to the main bars is 1¾ in.

The concrete was a 1 : 1½ : 3 mixture, with a nominal water-cement ratio of 0·5, using ordinary Portland cement. The materials were measured by weight and the concrete vibrated. The average crushing strength at an age of seven days was 3600 lb. per square inch and at six weeks about 6000 lb. per square inch.

The architects are Messrs. Taylor & Young, F./F.R.I.B.A., and the structural design and construction were by Messrs. Matthews & Mumby, Ltd., under the direction of their chief engineer, Dr. D. D. Matthews, M.A., D.Eng., who also evolved the method (known as the "method of balancing curvatures") used for calculating the bending moments on the girders.

Back Numbers of

"Concrete and Constructional Engineering"

The Publishers frequently have enquiries for back numbers of "Concrete and Constructional Engineering," generally from universities and similar institutions abroad. Readers who have copies published before 1954 (either bound or unbound) which they no longer require are invited to send a list of the dates to the Publisher, Concrete Publications, Ltd., 14 Dartmouth Street, London, S.W.1. Generally at least the published price can be offered for such back numbers.

A Large Store for Fertilizers.

THE store shown during construction in *Figs.* 1 and 2 has recently been built in Durban, South Africa, for Windmill Fertilizers, Ltd. The fertilizer is delivered to a railway siding, situated under cover to the left of the store, from whence it is carried by a vertical conveyor to the apex of the store and then by a horizontal conveyor along the building. From this conveyor the fertilizer may be discharged where required. The storage capacity of the building is about two-thirds of the total internal volume. The store is 195 ft. long by 148 ft. wide and has a total height of 61 ft. 4 in.

The sloping side walls are 23 ft. high and are formed of precast panels, 4 in. thick by about 1 ft. wide, set in grooves in the top and bottom members of an in-situ frame.

The gable walls are vertical. The lower portion is of concrete to match the sloping side walls ; above this the central portion is glazed, and on each side is a 9-in. brick wall extending to the roof and framed with reinforced concrete columns.

The three-pin arch ribs forming the upper part of the structure are at 10 ft. centres and are supported on inclined columns, which are part of the frame of the sloping walls. The ribs are 10 in. wide and taper in depth from 1 ft. 9 in. at their mid-points to 1 ft. at each end. The lower end of each rib is rounded to form a hinge and the upper end is shaped to receive a cast-iron shoe.

The ribs were precast on a segmental platform (in the foreground of *Fig.* 1). An average rate of production was four a day, and they were left for 42 days before they were removed from the centering. The ribs were lifted by three mobile cranes and a mast 100 ft. high with blocks and tackle and a hand-operated winch. One of the cranes lifted the ribs from the casting platform and transported them to the required position on the floor of the store. Each rib was then lifted by two mobile cranes, placed on the inclined columns, and then slung from the mast (*Fig.* 1). Each pair of ribs was joined by a cast-iron shoe, and precast concrete purlins were fixed to brace the arches to the previously-erected gable wall or to the



Fig. 1.—View during Construction.



Fig. 2.—The Store during Construction.

adjacent ribs. The total weight of two ribs forming an arch was about 21½ tons and the rate of erection was one arch per day.

The roof is covered with corrugated asbestos-cement sheets, which were purpose-made, fixed with hook-bolts to steel angles. The building was designed so that it may be extended, and the temporary end is covered with flat asbestos-cement sheets fixed vertically with hook-bolts to steel angles.

The concrete mixture for the arches, columns, and walls was 1 : 1 : 2 with aggregate of ¾ in. maximum size.

The architect was Mr. Paul H. Connell and the consulting engineer the late F. H. Preijer. The contractors were Messrs. Rush & Tompkins (S.A.), (Pty.), Ltd.

Effect on Beams of Static and Dynamic Loads.

In a paper presented to the American Concrete Institute recently, Mr. Joseph Penzien and Mr. Robert J. Hansen describe tests made on reinforced concrete beams to determine their behaviour when subjected to static and dynamic loads. The authors' conclusions are as follows.

(1) The effective strains in the concrete of beams subjected to static loading may be calculated with reasonable accuracy by the straight-line theory, provided that the beams contain compressive reinforcement. If no compressive reinforcement is used, strains may be considerably larger than those given by this theory. (2) The maximum strains in a beam subjected to a dynamic load may be considerably larger than those in the same beam when a static load of equal magnitude is applied. (3) Internal damping existing in beams subjected to dynamic forces reduces the maximum strains considerably. (4) Internal damping which exists in beams has characteristics similar to the damping properties of liquids. (5) The damping coefficient or percentage of critical damping in the beams tested varied between 6 and 8 per cent. (6) The same damping coefficient applies reasonably well to all frequencies and loads. (7) The yield point of the reinforcement is considerably higher under dynamic loading conditions than under static loading conditions, provided that the rate of dynamic loading is sufficiently great. For rates of loading within the range $1{\cdot}2 \times 10^6$ and $8{\cdot}5 \times 10^6$ lb. per square inch per second, standard reinforcement (A.S.T.M. Designation A15–48) has a dynamic yield point about 35 per cent. higher than its static yield point. (8) A reinforced concrete structure designed to withstand dynamic loads should have a low percentage of reinforcement. (9) Both the dynamic magnification factor and increase in the yield stress of the reinforcement should be considered.

The Design of Staircases.

By A. KRYSZTAL, A.M.I.E. (Australia), A.M.I.Struct.E.

IN the common method of designing staircases it is not usual to consider that the flight-slab is supported at its intersection with the walls or by cranked beams, or to allow for the effects of thrusts at the intersections of inclined slabs. The methods described in the following take account of supports on three sides of a slab and also the support provided by the intersection of the flights with the landings.

In *Fig.* 1 are a plan and section of a staircase which will be used as an example. The usual method is to provide a simple support along lines a–i and d–m and a slab spanning in one direction. Where the span exceeds the practical limit a beam is provided along lines b–k or c–l, or beams may be introduced along both lines to form a slab on four supports and spanning in two directions. In the latter case the beams are supported at b, k, c, and l. It is possible to dispense with these beams, which are often undesirable from an architectural point of view, by increasing the thickness of the landing slabs a–b–i–k and c–d–l–m and providing additional supports along lines a–b, c–d, i–k, and l–m in the form of bearing walls or cranked beams around the well. Generally there is an existing support along lines b–c and k–l which is not taken into account in the ordinary method of design, a straight joint being assumed.

The following method of calculation takes into account the supports along b–c and k–l. This means that the slab, having a well in the middle, is supported along all four external edges. For simplicity of calculation the slab is divided into two landing-slabs a–b–i–k and c–d–l–m, and two flight-slabs a–d–e–h and e–h–i–m. Each component forms a slab supported along three edges and free at the fourth. The calculation of the moments is facilitated by the coefficients given by Professor S. Timoshenko in "Theory of Plates and Shells." These coefficients are given in *Fig.* 2 for cases in which three edges are simply supported and in which one edge is fixed and two edges simply supported. The coefficients in *Fig.* 3 for the distribution of the load along the edges are from "Reinforced Concrete," by A. Suchnowski (Moscow, 1940).

Method of Calculation.

LANDING-SLAB.—The total load is 173 lb. per square foot, comprising: Slab (assumed to be 5 in. thick), 63 lb.; finishes 10 lb.; and live load 100 lb. This slab is actually part only of a whole staircase slab supported on four edges. The load is therefore divided in two directions in accordance with the ratio of the width to the length of the whole slab. Neglecting the opening for the well, which has only small influence on the distribution of the load, the best approximate method of finding the proportion of the load in either direction is by the Grashoff-Rankine coefficients given in "Reinforced Concrete Designer's Handbook," by Chas. E. Reynolds, and which are plotted in *Fig.* 4.

The ratio of the length to the width of the staircase-slab is $\frac{W}{L} = \frac{11}{20{\cdot}08} = 0{\cdot}55$, and the portion of the load on the landing-slab, in accordance with *Fig.* 4, is $0{\cdot}92 \times 173 = 159$ lb. per square foot. The ratio of the length to the width of

the landing-slab only is $\frac{5}{11} = 0{\cdot}45$, and the bending moments in each direction, in accordance with *Fig.* 2, are

$M_x = 0{\cdot}005 \times 159 \times 11^2 = 1060$ ft.-lb. and $M_y = 0{\cdot}02 \times 159 \times 11^2 = 380$ ft.-lb.

For an allowable compressive stress in the concrete of 875 lb. per square inch, a tensile stress in the steel of 18,000 lb. per square inch, and $m = 12$, the minimum

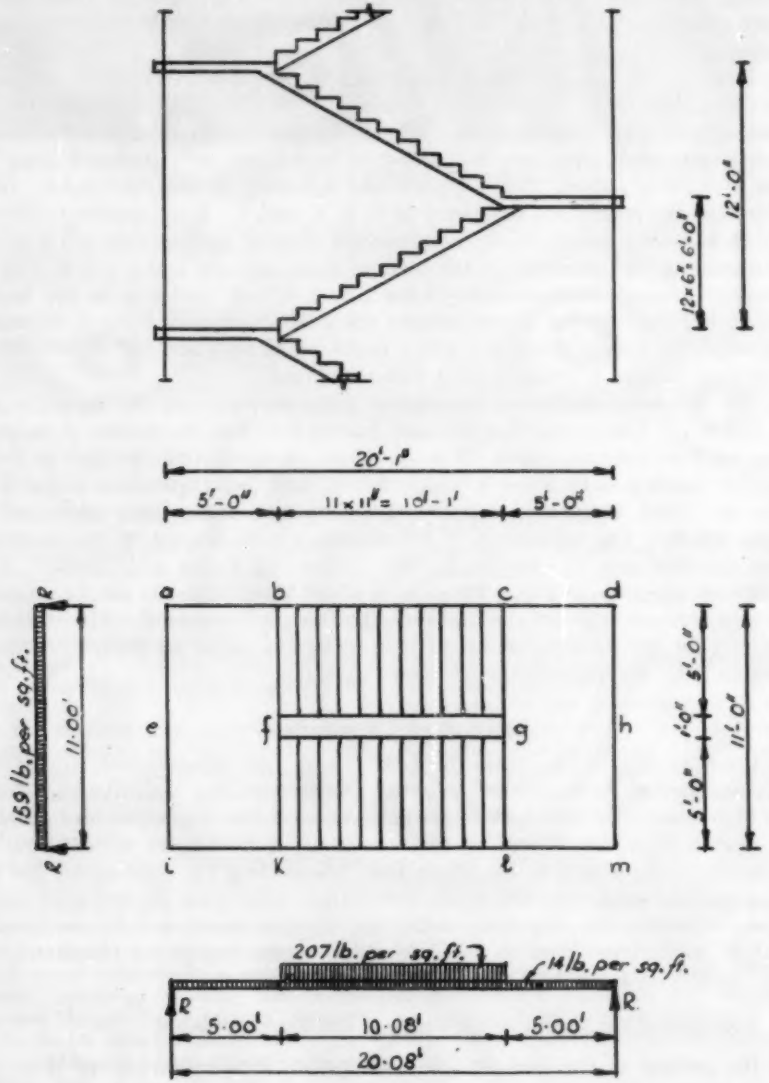


Fig. 1.

effective depth of the slab is $d = \sqrt{\frac{1060}{141}} = 2{\cdot}74$ in. The reinforcement required in the two directions is $A_x = 0{\cdot}193$ sq. in. ($\frac{3}{8}$-in. bars at 6-in. centres), and $A_y = 0{\cdot}079$ sq. in. ($\frac{1}{4}$-in. bars at 6-in. centres).

FLIGHT-SLAB.—The load on the sloping part of the flight-slab with an angle α, where $\tan \alpha = \frac{6}{11} = 0{\cdot}545$, and therefore α is 28·5 deg. and cos α is 0·879, is

Weight of steps, $\frac{1}{2} \times \frac{11 \times 6}{144} \times 140 \frac{12}{11}$	= 35 lb. per sq. ft.	
,, ,, 5-in. sloping slab, $\frac{62{\cdot}5}{0{\cdot}879}$	= 71	,, ,, ,, ,,
Finishes	= 15	,, ,, ,, ,,
Live load	= 100	,, ,, ,, ,,
Total	= 221	,, ,, ,, ,,

The distribution of the load on the full length of the flight-slab is shown in *Fig.* 1.

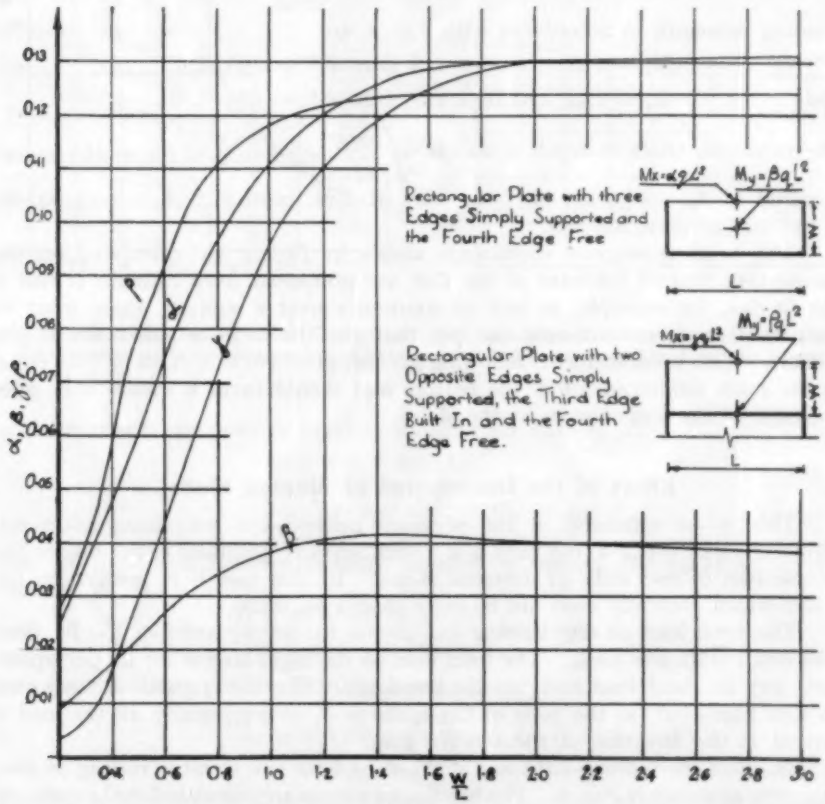


Fig. 2.—Bending Moments on Slabs Supported on Three Sides Only.

As the coefficients for the bending moments given in *Fig.* 2 relate to a uniformly-distributed load, it is necessary to adjust them by a factor K which takes into consideration the actual distribution of loads. This factor is assumed to be about equal to the ratio of the maximum bending moment due to the actual loads on a simply-supported slab to the maximum bending moment for the same total load uniformly distributed over the full length of a slab.

The reactions due to the actual load on a simply-supported slab are

$$R = 14 \times 10{\cdot}04 + 207 \times 5{\cdot}04 = 1180 \text{ lb.},$$

and the maximum bending moment

$$M = 1180 \times 10{\cdot}04 - 14\frac{10{\cdot}04^2}{2} - 207\frac{5{\cdot}04^2}{2} = 8510 \text{ ft.-lb.}$$

For the same load uniformly distributed the maximum bending moment is $M_{\cdot} = \frac{2 \times 1180 \times 20{\cdot}80}{8} = 5920$ ft.-lb. K is therefore $\frac{8510}{5920} = 1{\cdot}44$.

The ratio of the spans of the flight-slab is $\frac{5}{20{\cdot}08} = 0{\cdot}25$ and the respective bending moments, in accordance with *Fig.* 2, are

$$M_x = 1{\cdot}44 \times 0{\cdot}03 \times 118 \times 20{\cdot}08^2 = 2055 \text{ ft.-lb.}$$

and

$$M_y = 1{\cdot}44 \times 0{\cdot}01 \times 118 \times 20{\cdot}08^2 = 685 \text{ ft.-lb.}$$

The minimum effective depth of slab is $\sqrt{\frac{2055}{141}} = 3{\cdot}8$ in. and the reinforcement required is $A_x = 0{\cdot}39$ sq. in. ($\frac{1}{2}$-in. bars at 6-in. centres); $A_y = 0{\cdot}15$ sq. in. ($\frac{3}{8}$-in. bars at 9-in. centres).

The bending-moment coefficients shown in *Fig.* 2 are calculated on the assumption that the corners of the slab are prevented from lifting. If this is not so due, for example, to lack of continuity over a support, there must be additional reinforcement near the top to resist the torsional moments at the corners. This reinforcement, following general practice, is a mesh of bars equal to the main reinforcement in the bottom and should cover a square with sides one-fifth of the longer span of the slab.

Effect of the Intersection of Sloping Slabs.

This is an extension of the previous method but introduces additional supports along b–f, f–k, c–g, and g–l. Such support generally exists due to the intersection of two slabs of different slopes. In this case it is possible to use a slab which generally need not be more than 4 in. thick.

The total load on the landing-slab is 160 lb. per square foot (60 lb. dead load and 100 lb. live load). The total load on the flight-slab is 207 lb. per square foot (107 lb. dead load and 100 lb. live load). The landing-slab is supported on four edges but, as the ratio of the spans is $\frac{5}{11} = 0{\cdot}45$, nearly all the load is carried in the direction of the shorter span.

In effect the landing-slabs and flight-slabs form one member resting on four supports, as shown in *Fig.* 5. The bending moments are calculated in the ordinary way and adjusted by the factor K to allow for the support along k–l.

The general formula for negative bending moments at supports k-f, f-b, l-g, and g-c, is

$$M = \frac{w_1 l_1^3 + w_2 l_2^3}{4(2l_1 + 3l_2)} = \frac{160 \times 5^3 + 207 \times 10{\cdot}08^3}{4(2 \times 5 + 3 \times 10{\cdot}08)} = -1440 \text{ ft.-lb.}$$

For the maximum positive bending moment on span l_1 the negative bending moment is

$$M = -\frac{160 \times 5^3 + 107 \times 10{\cdot}08^3}{4(2 \times 5 + 3 \times 10{\cdot}08)} = -800 \text{ -ft.,lb.}$$

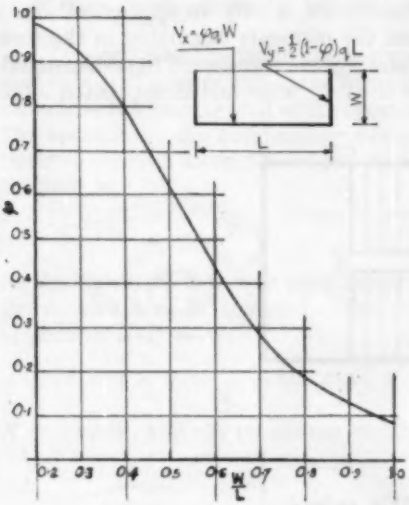


Fig. 3.—Distribution of Load along the Supports of a Slab Supported on Three Sides only.

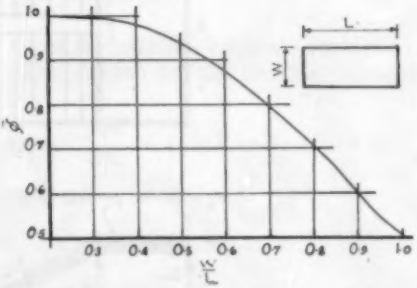


Fig. 4.—Grashoff-Rankine Coefficients for Proportion of Load on the Shorter Span.

and the maximum positive bending moment on span l_1 is

$$\frac{\left(\frac{160 \times 5}{2} - \frac{800}{5}\right)^2}{2 \times 160} = 180 \text{ ft.-lb.}$$

For the maximum positive bending moment on span l_2 the negative bending moment is

$$M = \frac{60 \times 5^3 + 207 \times 10{\cdot}08^3}{4(2 \times 5 + 3 \times 10{\cdot}08)} = -1360 \text{ ft.-lb.,}$$

and the maximum positive bending moment is

$$\frac{207 \times 10{\cdot}80^2}{8} - 1360 = 2630 - 1360 = 1270 \text{ ft.-lb.}$$

The factor K is here $\frac{1270}{2630} = 0{\cdot}48$ and, as the ratio of the spans of the flight-slab

is $\frac{5}{10{\cdot}08} = 0{\cdot}5$, the positive bending moments on the flight-slab allowing for support along the third side are (*Fig.* 2)

$$M_x = 0{\cdot}48 \times 0{\cdot}06 \times 207 \times 10{\cdot}08^2 = 610 \text{ ft.-lb.};$$
$$M_y = 0{\cdot}48 \times 0{\cdot}022 \times 207 \times 10{\cdot}08^2 = 220 \text{ ft.-lb.}$$

A similar adjustment is made to the negative bending moment:

$$M = -0{\cdot}48 \times 1440 = -690 \text{ ft.-lb.}$$

It must, however, be pointed out that the reduction of the bending moments by K, particularly the negative bending moments, is only an approximation.

A slab 4 in. thick is sufficient to resist the moments calculated in the foregoing. It remains to show how loads *Rkf* and *Rgl* are transferred to the surrounding walls or beams. As the landing-slab is 11 ft. wide and is supported along

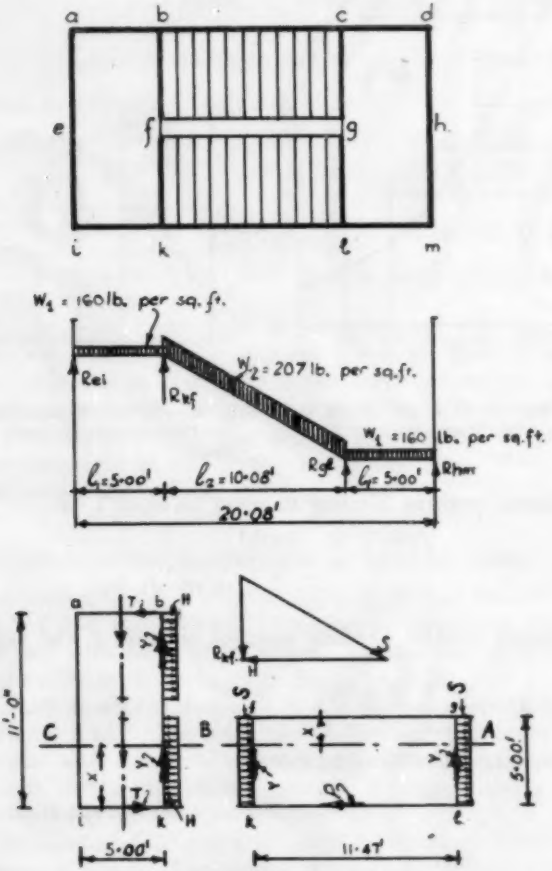


Fig. 5.

10 ft. only, the maximum load from the landing-slab is

$$R'kf = 1{\cdot}1\,\frac{160 \times 5}{2} + \frac{690}{5{\cdot}00} = 580 \text{ lb. per foot.}$$

The maximum load of the flight-slab, for a ratio of spans of 0·5 (*Fig.* 3), is $R''kf = \frac{1}{2}(1 - 0{\cdot}62)207 \times 10{\cdot}08 = 400$ lb. per foot. Therefore

$$Rkf = 580 + 400 = 980 \text{ lb. per foot.}$$

Load Rkf can be resolved into two directions parallel to the adjacent slabs, giving

$$H = \frac{980}{0{\cdot}545} = 1800 \text{ lb. per foot and } S = \frac{1800}{0{\cdot}879} = 2050 \text{ lb. per foot.}$$

The load S on the flight-slab is considered first. As shown in *Fig.* 5, a similar load acts at the lower end of the flight-slab f–k–g–l and therefore the total of both loads S on the bearing wall or the cranked beam is $P = 2 \times 5 \times 2050 = 20{,}500$ lb. The reaction to the overturning moment due to loads S and the force P causes shearing stresses along the intersection of the flight-slab and landing-slab. This reaction is

$$Y = \frac{20{,}500 \times 2{\cdot}5}{11{\cdot}47 \times 5} = 900 \text{ lb. per foot.}$$

As the flight-slab is now in equilibrium so far as the external loads are concerned, the stresses may be analysed. The general expression for the bending moment at section A–B is

$$M = 2 \times 2050\frac{X^2}{2} - 900 \times X \times 11{\cdot}47 \text{ and } \frac{dM}{dX} = 4100X - 10{,}320 = 0.$$

$X = 2{\cdot}52$ ft., and the maximum bending moment is

$$M_{max.} = 4100 \times \frac{2{\cdot}52^2}{2} - 10{,}320 \times 2{\cdot}52 = -13{,}000 \text{ ft.-lb.}$$

The modulus of section of the slab, ignoring the reinforcement, is

$$Z = \frac{4 \times (12 \times 11{\cdot}47)^2}{6} = 12{,}510 \text{ in.}^3$$

The maximum stresses are $\pm \dfrac{13{,}000 \times 12}{12{,}510} = \pm 12{\cdot}5$ lb. per square inch, which can be resisted by plain concrete.

The landing-slab is acted upon by two pairs of loads in opposite directions. The first pair is $H = 1800$ lb. per foot acting perpendicularly to the centre-line of the slab, which will accordingly function as a deep beam. The second pair is $Y = 900$ lb. per foot acting parallel to the centre-line of the slab. The reaction is

$$T = \frac{1800 \times 5 \times 6}{11} - \frac{2 \times 900 \times 5 \times 2{\cdot}5}{11} = 2860 \text{ lb.}$$

The general expression for the bending moment at a section B–C is

$$M = 2860X - 1800\frac{X^2}{2} + 900 \times 2{\cdot}5 \times X;$$

$$\frac{dM}{dX} = 2860 - 1800X + 900 \times 2{\cdot}5 = 0.$$

Therefore $X = 2{\cdot}84$ ft., and the maximum bending moment is

$$M_{max.} = 2860 \times 2{\cdot}84 - 1800\frac{28{\cdot}4^2}{2} + 900 \times 2{\cdot}85 \times 2{\cdot}5 = 7250 \text{ ft.-lb.}$$

The modulus of section of the slab, ignoring the reinforcement, is

$$Z = \frac{4 \times (12 \times 5)^2}{6} = 2400 \text{ in.}^3,$$

and the maximum stresses are

$$\frac{900 \times 5}{4 \times 12 \times 5} \pm \frac{7250 \times 12}{2400} = 19 \pm 36 = -17 \text{ lb. and } +55 \text{ lb. per square inch.}$$

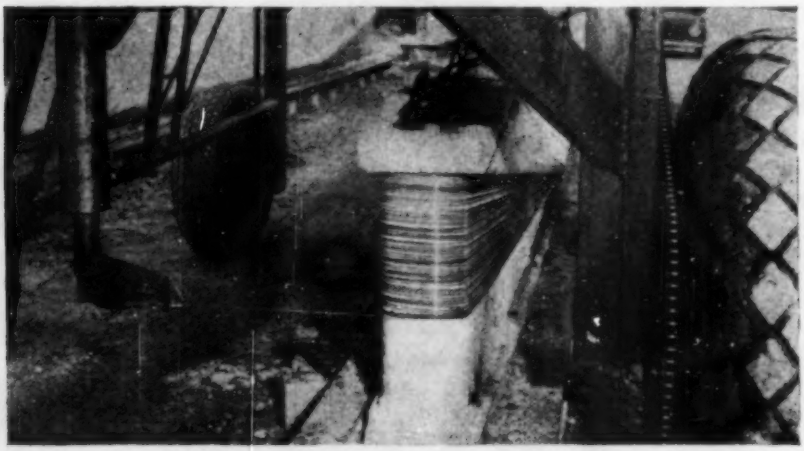
These stresses also can be resisted by plain concrete.

The author is indebted to Mr. J. E. Delprado for helpful criticism.

New Method of Prestressing Beams.

A METHOD of prestressing beams in a manner similar to that used for tanks has been devised by the Crom Corporation, of Gainesville, Florida, U.S.A. The illustration shows a beam 48 ft. long being prestressed by this method.

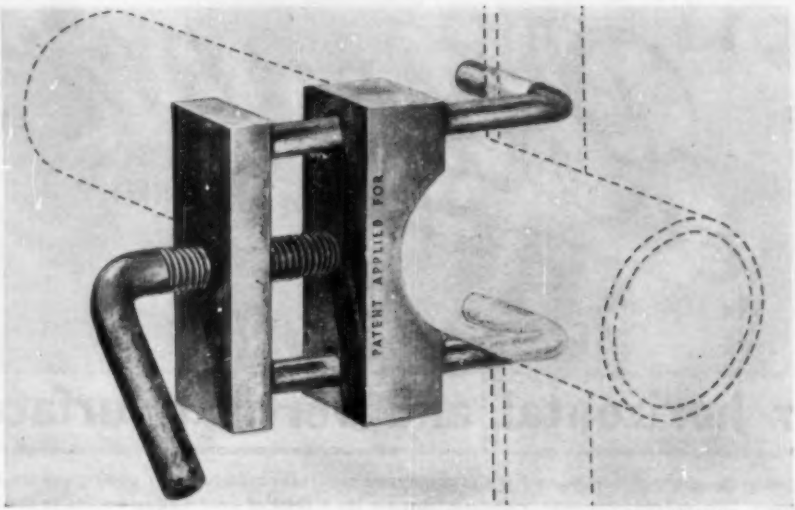
The wire is 0·14 in. diameter. A coil of wire weighing 400 lb. is mounted on a 4-wheel petrol-driven chassis which straddles the beam. Below the chassis the wire passes through a die which is moved from one side of the beam to the other as the vehicle travels back and forth along the length of the beam. In the case of the beam illustrated, which is one of a number made for a floor of an office building, there are 92 loops of wire tensioned to 140,000 lb. per square inch. The wire is anchored at one end by a strip of steel, and is covered by a layer of lightweight concrete placed pneumatically. The beams were designed and prestressed as tee-beams, and the top flange was added when the floor slab was concreted.



Looping Wires under Tension around a Beam.

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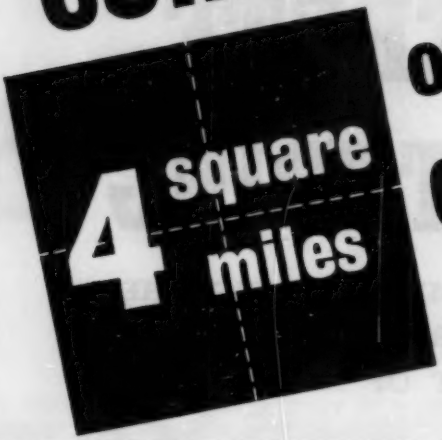
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Test of Prestressed Beam of 55 ft. Span.

INTERPRETATION OF TEST RESULTS FROM LOAD-DEFLECTION CURVES.

A PRESTRESSED beam of 55 ft. span made for the roof of a channel constructed across Leyton Marshes, London,* has been tested to destruction. The test was made 48 days after the beam had been cast. The beam was cast in two sections, about 19 ft. and 38 ft. long, which were joined with mortar and stressed with three 1⅛-in. diameter high-tensile bars. The section (*Fig.* 1) was I-shape, 2 ft. 6 in. deep and 1 ft. 11¾ in. wide, with five full-width stiffeners at intervals along the length. The I-section was changed to a rectangle at each end. The effective area (A) of the section was 280¼ sq. in., the effective moment of inertia (I) was 36,155 in.⁴, the eccentricity (e) of the prestressing bars at mid span was 11·6 in., the dead-load moment (M_D) was 1,470,000 in.-lb., and the live-load moment (M_L) was 3,855,000 in.-lb.

The beam was simply supported on a span of 55 ft. and the load applied at two points 3 ft. on each side of the centre of the span by an hydraulic jack and distributing beam. The beam was loaded gradually until the stresses at the centre of the span were slightly in excess of their design value and the load then released. A load 50 per cent. greater was then applied in a similar manner and removed. Finally the beam was loaded until it broke.

The beam broke when the applied load was 27·9 tons, the bending moment due to this load being 9,200,000 in.-lb. at midspan. The dead-load moment was 1,470,000 in.-lb. and the total bending moment 10,670,000 in.-lb.

The following expression was used to determine the theoretical bending moment at failure (M_{ult}):

$$M_{ult} = A_s f_{ult}\left(1 - 0{\cdot}5p\frac{f_{ult}}{C_r}\right)d,$$

where A_s is the area of steel (three bars of 1⅛ in. diameter = 2·98 sq. in.); f_{ult} is the breaking strength of the bars (assumed to be 150,000 lb. per square inch); p is the proportion of steel to the area of the rectangle formed by the full width of the beam and the depth to the steel (0·00465); C_r is the compressive strength in bending of the concrete (6800 lb. per square inch); and d is the depth of the steel from the top flange (27 in.). The theoretical bending moment that would cause failure, calculated from the expression, was 11,450,000 in.-lb., that is 7·3 per cent. more than the actual bending moment at failure. The factors of safety were: live load (S_L) $\dfrac{M_{ult} - M_D}{M_L}$ and live load plus dead load (S_{L+D}) $\dfrac{M_{ult}}{M_L + M_D}$. The results obtained were $S_L = 2{\cdot}4$ and $S_{L+D} = 2$ for these factors.

The relation between the load and the deflection at the centre is shown in *Fig.* 2 where the results of the three loadings are plotted. From the slope of the linear portion of these curves the stiffness of the beam may be determined to be 14·55 tons per inch of deflection. From the support and load conditions and the properties of the section an effective modulus of elasticity (E) may be determined as $5{\cdot}26 \times 10^6$ lb. per square inch.

For the reasons given later the load-deflection curves indicate that between zero tensile stress and cracking of the concrete a load of 2·1 tons was applied corresponding to an increase in bending moment of 690,000 in.-lb. The tensile strength of the concrete in bending calculated from this moment was 290 lb. per square inch; this low result was thought to be due to the joint at the one-third point opening before the concrete had cracked at the centre of the span.

The applied bending moment at the point where the curve ceases to be linear in the second loading coincides with the design-load bending moment of 3,885,000 in.-lb. The expression for the value of the prestressing force H may be used to obtain its value:

$$H = \frac{M_D + M_L}{e + \dfrac{I}{Ay^2}} = 262{,}500 \text{ lb.}$$

As the three bars had an area of 2·98 sq. in. they were stressed to 39·3 tons per square inch under the full design-load bending moment, and the compressive

* See this journal for October, 1952.

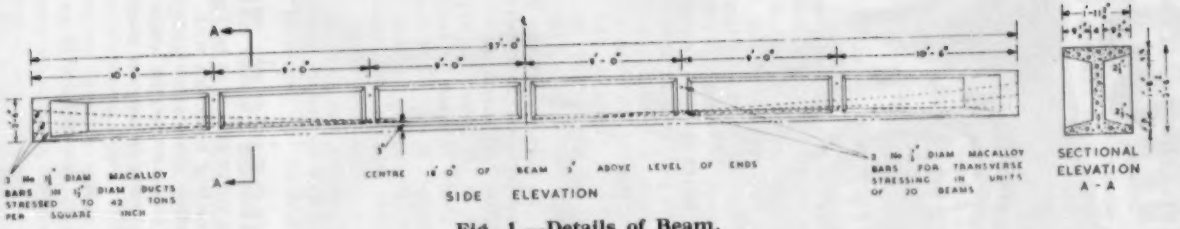


Fig. 1.—Details of Beam.

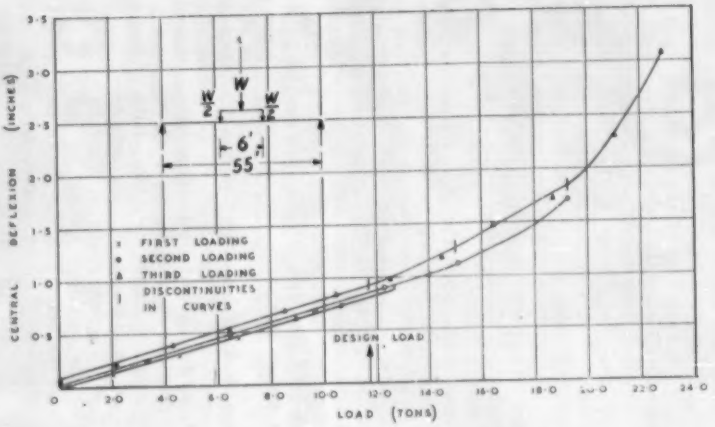


Fig. 2.—Deflections at Different Loads.

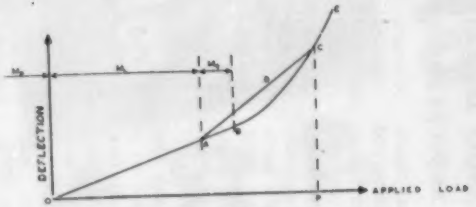


Fig. 3.—Load-deflection Curve.

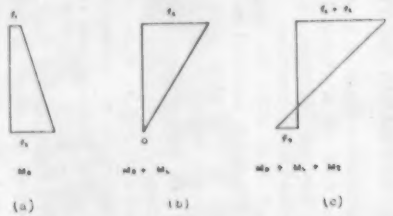
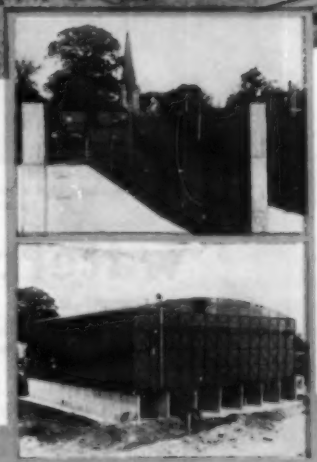


Fig. 4.—Stresses at Three Loadings.



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Some views of the open-air swimming pool at the Skegness Holiday Camp. By kind permission of Messrs. Butlins Ltd.

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stress in the concrete was 1920 lb. per square inch.

Assuming that the prestress remained unaltered with the reduction of load, the compressive stresses due to dead load only were 280 lb. per square inch at the top and 1560 lb. per square inch at the bottom.

Since the beam was supported at the time of prestressing in such a way that the dead-load bending moment was only 876,000 in.-lb., the change in stress in the steel was 3·5 tons per square inch. As the initial stress in the bars was 42 tons per square inch, the loss of stress was

$$\frac{42 - (39{\cdot}3 - 3{\cdot}5)}{42} \times 100 = 14{\cdot}5 \text{ per cent.}$$

Bond tests were made of the bars at each end of the beam where lengths of about 3 ft. were undamaged; the average force required to pull out the bars was 32 tons. The grout consisted of two parts of rapid-hardening Portland cement to one part of water. The materials were mixed by hand and poured through a fine sieve, in order to exclude lumps, into the pressure vessel. The holes in the beam were cleaned by passing water through them before the grout was forced through at a pressure of 50 lb. per square inch.

Information derived from Load-deflection Curves.

The Cement and Concrete Association, by whom the test was made, has supplied the following notes.

The load-deflection curve generally takes the form shown in *Fig.* 3. The first loading up to a load P gives the curve OABC and a second loading to a load greater than P follows the curve OADCE. The change at B from the straight line OAB to the curve BC is attributable to the tensile strength of the concrete being exceeded and cracks being formed which reduce the stiffness of the beam. The stress at B is shown in *Fig.* 4 (*c*), where f_t is the tensile strength in bending of the concrete.

On the second loading the cracks re-open at A when the stress at the bottom of the beam exceeds zero, since the tensile resistance has previously been destroyed. The stress condition at A is shown in *Fig.* 4 (*b*).

The difference in load between points A and B and the form of loading enable the moment M_t to be determined. Since $\frac{M_t}{z} = f_t$, the tensile strength of the concrete in bending may be calculated.

When the beam is in the loaded condition at A the centre of pressure of the prestressing force must be at the upper limit of the core, which is $\frac{I}{Ay_2}$ (where y_2 is the distance from the centroid to the bottom of the beam). The effect of the applied moment $M_D + M_L$ has thus been to displace the centre of pressure from the position of the cable, with eccentricity e, to $\frac{I}{Ay_2}$. Therefore

$$\frac{M_D + M_L}{H} = e + \frac{I}{Ay_2}$$

and the prestressing force

$$H = \frac{M_D + M_L}{e + \frac{I}{Ay_2}}.$$

The compressive stress in the concrete is $\frac{y_2 H}{dA}$, and the initial stresses f_1 and f_2 [*Fig.* 4 (*a*)] are found by subtracting the stresses due to M_L from the stresses shown in *Fig.* 4 (*b*).

Film on Prestressed Concrete.

A 16-mm. film showing the construction of a railway goods shed at Bury St. Edmunds (illustrated in our number for May, 1954) has been made by Mr. Charles R. Price, the main contractor who also carried out the post-tensioned work. The pre-tensioned beams were made by Anglian Building Products, Ltd. The film is available on loan from Mr. Charles R. Price, of Kingsbury Road, London, N.W.9.

Book Reviews.

"Prestressed Concrete." By Gustave Magnel. Third Edition. (London: Concrete Publications, Ltd. 1954. Price 20s.)

THE third edition of this well-known book has been brought completely up to date by the inclusion of a considerable amount of new matter. Since the second edition was published in 1950 many new systems have been introduced and most of these are now described, but it is noticeable that only one method considered worthy of inclusion has been developed in this country. Those concerned with the design of prestressed beams will welcome two of the additions. One is a table giving the dimensions of beams for various spans and loads together with the number of cables required. This is of considerable use in simplifying the choice of the approximate dimensions of a beam. Another useful addition is a method of calculating the ultimate strength of beams. Compared with many such methods that have been presented this is noteworthy for its simplicity, and is based on the results of tests made by the author on beams of sizes in common use. The method may be applied to rectangular beams or to beams with flanges. Further notes on continuous structures include data on the economical spans for continuous beams and on their breaking strength.

Professor Magnel believes very strongly that the uncertainties which exist in the design of structural members will not be resolved by testing tiny specimens, and the many tests described have been carried out on full-size beams and slabs. Details are given of fatigue tests, of the resistance of beams to extremes of temperature and to dynamic loads, of a test to failure of a slab prestressed in two directions, and of a mast in the form of a Vierendeel girder, as well as the results previously given which include those relating to a continuous beam and a freely-supported beam of 155 ft. span.

It is, however, to the chapter on applications of prestressed concrete that most has been added and this, of course, is a measure of the success which has been achieved in applying the principle of prestressing to all types of structures. Many new bridges are described, including one recently built over the river Rhine, without the use of staging, at a position where the river is more than 1000 ft. wide. Included also are descriptions of building frames, and a flat-slab floor designed to carry very heavy loads. The examples are taken from all parts of the world and are not confined to applications of the Belgian method.

This book is too well known to require recommendation; its value to those specializing in prestressed concrete as well as to those requiring an easily understood text-book and source of reference may be gathered from the fact that this is the third edition since 1948.—J. E. G.

"Soil Survey Procedure." Road Research Technica Paper No. 15. (H.M.S.O. Price 1s. 9d.)

A FEW years ago, before the craze for changing well-established terms, this paper would have been called "Site Investigation." It describes methods of obtaining samples of subsoils by means of hand-operated and power-driven augers, and of ascertaining the depth of ground water. Other sections deal with the classification of soils and the presentation of the information gained.

"Hochbauten der Eisenbahn." By R. Spröggel. (Berlin: Springer-Verlag. Price 31.50 D.M.)

DEALS with the design and planning of railway stations and other structures used on railways in Germany.

Books Received.

"O Processo da Compensação dos Deslocamentos." By Pedro B. J. Gravina. (São Paulo, Brazil: Escola Politécnica da Universidade de São Paulo. Price not stated.)

"Effect of Edge Insulation upon Temperature and Condensation on Concrete Floor Slabs." By Harold R. Martin, Paul R. Achenbach, and Richard S. Dill. (Washington, U.S.A.: U.S. Government Printing Office. Price 20 cents.)

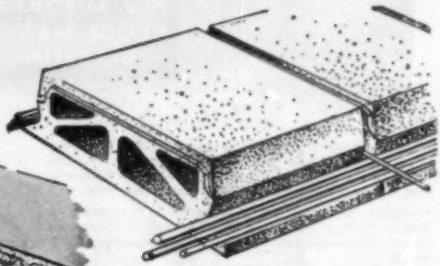
"Circular Plates Subjected to Radially Symmetrical Transverse Load Combined with Uniform Compression or Tension in the Plane of the Plate." By Ove Petersson. (Stockholm: Teknisk Tidschrifts Förlag. Price 7 Kr.)

"Modulação Das Construções. Exemplos De Aplicação." By Alberto José Pressoa. **"A Mecânica Dos Solos: Seu Papel Na Engenharia Civil."** By Manuel Rocha. [Lisbon: Laboratorio Nacional de Engenharia Civil (Ministério das Obras Públicas.) Prices not stated.]

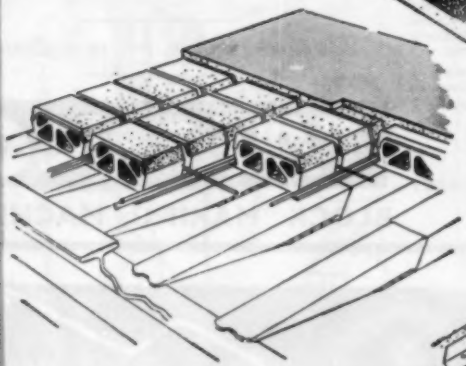
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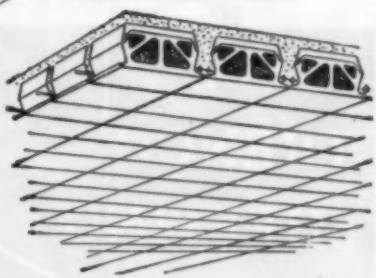
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Television Transmission Towers.

THE tower shown in *Fig.* 2 is one of a number at intervals of about 45 miles used for relaying television services between Hamburg and Frankfurt. Two types of towers are used, one about 227 ft. high and the other about 127 ft. high (*Fig.* 1).

The taller towers have an external diameter of 26 ft. 3 in. and at the top there are four floors, three of which and the roof project about 10 ft. beyond the circumference of the tower and the lowest floor projects about 4 ft. The smaller towers have a diameter of 23 ft.; the upper two floors and the roof project about 10 ft. and the lowest of the upper floors projects about 4 ft. only. The foundation is an annular footing. In the smaller towers the staircase is around the inside of the wall whereas in the larger towers an enclosed stairwell is provided.

The towers were constructed with sliding shutters. The walls are 7 in. thick and reinforced on both faces. Chases were formed in the walls so that the floors could be built after the walls were completed.

The imposed load on the upper floors

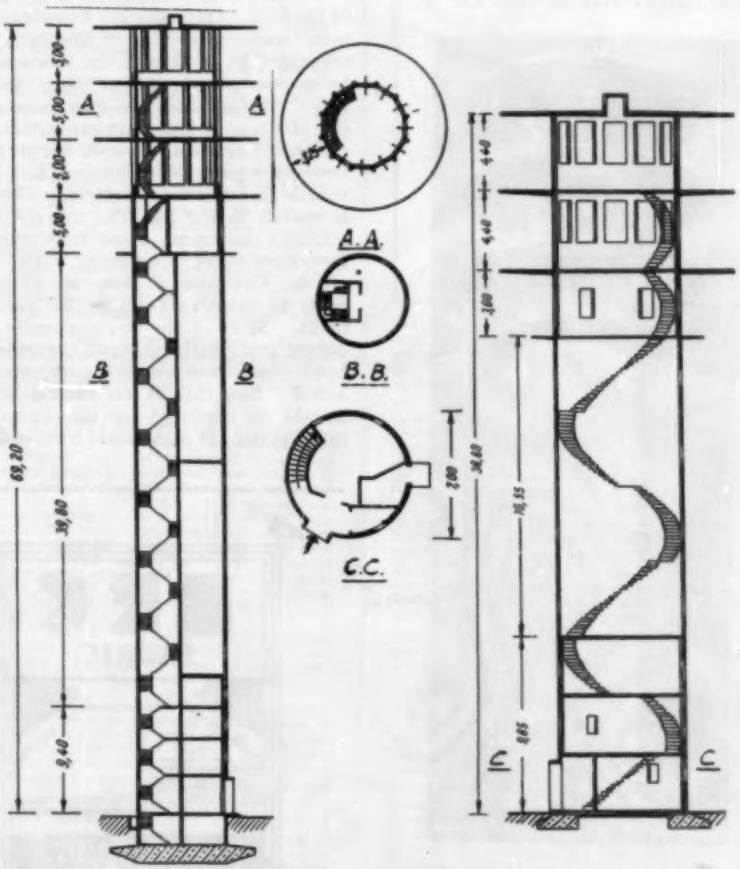


Fig. 1.—Sections (Dimensions in metres.)

is 123 lb. per square foot, and for the larger towers the total load on the foundation is about 3000 tons, of which 600 tons are due to imposed loads. The calculated horizontal deflections at the top of the larger towers is 1·7 in. due to wind, 0·5 in. due to eccentric loads on the upper floors, and 3·2 in. due to eccentric compression of the ground. The maximum bending moment, including the effect of wind, is about 21,700 ft.-lb., which, with the vertical load, produces a maximum stress in the wall of about 1140 lb. per square inch.

The foregoing is abstracted from "Beton- und Stahlbetonbau" for June 1953.

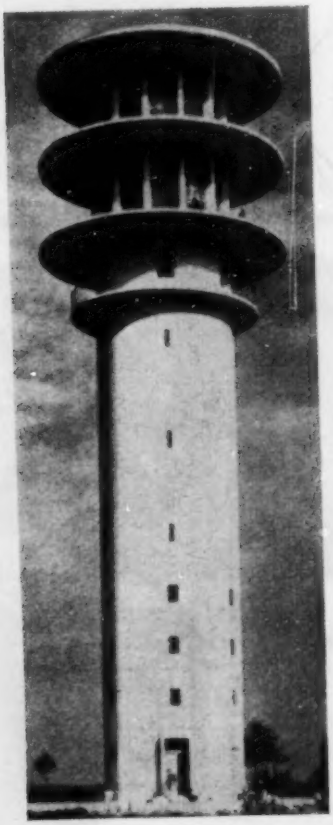


Fig. 2.

Joint-sealing Compounds.

BRITISH STANDARD No. 2499, "Tests to Assess the Properties of Hot Applied Joint Sealing Compounds for Concrete Pavements," has been issued by the British Standards Institution at a price of 4*s*. The compounds are classified according to their resistances to flow in hot weather and to the penetration of grit. Details are given of a method of sampling, and of tests for the determination of the "pour point," settlement, resistance to flow, resistance to penetration by impact, and extensibility. The components are classified in three groups for use in different circumstances.

The title of this document is an example of the folly of introducing American terms into this country, for it has been found necessary to explain in the foreword that by pavements are meant roads and runways. Presumably we may now expect our amateur language-improvers to call a pavement artist a sidewalk artist, a road sweeper a pavement sweeper, and a road vehicle a pavement vehicle. The Road Research Board has also started to call a road a pavement, so we may soon see a Pavement Safety Campaign, with "Death on the Pavement" posters and instructions to children to Keep off the Pavement. Most of the developments in the design and construction of concrete roads have come from abroad, particularly the U.S.A., but this is no reason why we should be confused by the unnecessary importation of American terminology.



MISCELLANEOUS ADVERTISEMENTS.

Situations Wanted, 3d. a word : minimum, 7s. 6d. Situations Vacant, 4d. a word : minimum, 10s. Other miscellaneous advertisements, 4d. a word : 10s. minimum. Box number 1s. extra. The engagement of persons answering these advertisements is subject to the Notification of Vacancies Order, 1952.

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(*Continued on page* lxii)

MISCELLANEOUS ADVERTISEMENTS.

(*Continued from page* lxi)

SITUATIONS VACANT. The Trussed Concrete Steel Co., Ltd., have a vacancy in their Birmingham office for an experienced reinforced concrete detailer. Five-days' week. Pension scheme. Apply in writing to the LOCAL MANAGER, Engineering Department, 133 Edmund Street, Birmingham, 3.

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SITUATION VACANT. Production manager required for a progressive precast concrete company in Surrey, with wide experience in the industry both in the technical field and in the control of labour. Products cover a wide range. Excellent opportunity for man with the above qualifications. Adequate remuneration. Replies will be treated in the strictest confidence. Reply to Box 4053, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

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SITUATIONS VACANT. Assistants required by well-established firm of consulting engineers in Harrow, Middx. Excellent opportunities for detailers, draughtsmen, and designer-draughtsmen requiring either established positions or more experience. Good wages and excellent office conditions. Five-days' week. Close to Met. Station. Write stating age, experience, and salary required to Box 4054, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

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SITUATIONS VACANT. Detail and/or design draughtsmen with experience of steel or concrete structures required by Birmingham consulting engineer. Some training given if necessary. Salary according to experience. Box 4056, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Senior technical representative required by nationally known company manufacturing precast concrete floor and roof beams, anxious to expand this department. Only men with experience and first-class knowledge of design, sales and fixing of these products will be interviewed, and salary will be commensurate with ability and experience. Superannuation scheme. Box 4057, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Research assistant. Building organisation requires young man with Honours Degree and some experience for applied research largely in the field of concrete and prestressed concrete both on sites and in factories. Preference for applicants interested in quality control applied to concrete, prestressed design, prefabricated techniques, new materials and processes, testing of materials and components, etc. Salary according to experience. Five-days' week. Good pension arrangements. Box 4058, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

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Reinforced Concrete Association.

At the annual lunch of the Reinforced Concrete Association, held in London last month, the principal topic dealt with by the President (Mr. H. G. Cousins) was the courses of training for supervisors of concrete work which (as stated in our Editorial Note) have been arranged by the City and Guilds of London Institute and elsewhere. The scheme was proposed by the Association, and had been endorsed by the Institution of Civil Engineers, the Royal Institute of British Architects, the Institution of Structural Engineers, the Federation of Civil Engineering Contractors, and the National Federation of Building Trades Employers. Courses of instruction are to be held at ten or twelve other centres. A certificate will be awarded to men who pass the course, and the certificate may also be secured by men who sit for the examination without attending one of the courses. The first examination is to be held in May, 1955. He urged the members of the Association to encourage suitable men to attend the courses and also to provide lecturers.

The gold medal of the Association for the year 1953 was awarded to Dr. P. Abeles for his contribution to the advancement of reinforced concrete.

The Council.

The Council for the year 1954–1955 is as follows.

President, Mr. C. H. S. Howkins (Howard Farrow, Ltd.). Vice-president, Mr. D. H. New (Holland & Hannen and Cubitts, Ltd.).

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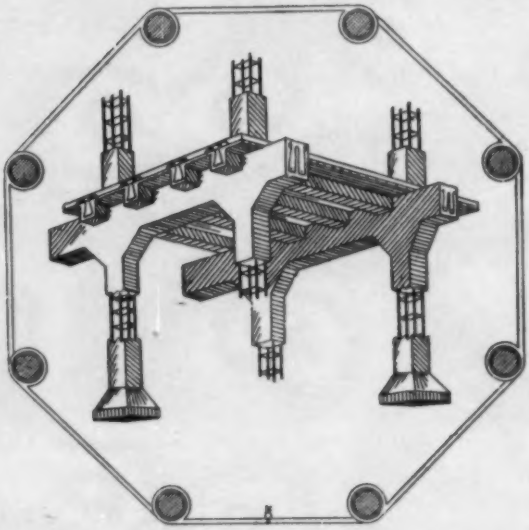
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